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DESIGN, MATERIAL AND WORKMANSHIP— A COMPLEX PROBLEM IN SHIP CONSTRUCTION

S. T. A. Janzén

Paper No. 1. Session 1975-76

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DESIGN, MATERIAL AND WORKMANSHIP—A COMPLEX PROBLEM IN SHIP CONSTRUCTION

By S. T. A. JANZÉN*

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1. INTRODUCTION

The Author's day-to-day work is mainly in ship plan approval, which today, is a fairly specialised and complicated subject. The field Surveyor, however, has to cope with many and varied problems regarding the quality of workmanship, materials used and standards adopted in the fabrication, assembly and construction of today's large ships.

Fig. 1 (ref. 1) lists the processes involved in the construction of steel ships. Each process or operation can malfunction and perhaps result in built-in defects which might later lead to structural failures. Of the items listed, the process of joining one member to another or one assembly to another is particularly important and will be dealt with in greater detail in later sections. The figure clearly indicates the enormity of the field Surveyor's task and for him to examine all such aspects of new construction without strong backing from the builder's own inspection and quality control teams, he would indeed need to be something of a superman.

No matter how good the standard of inspection, there will usually be variations in the capability of the final ship product. Some of the causes of this are illustrated in Fig. 2 (ref. 1) which shows further possible areas of defects in the materials themselves.

The size of the structure to inspect has increased enormously in recent years as shown in Fig. 3 (ref. 2). This increase in size has of necessity led to much improved load prediction and structural analysis methods. The demand for a quantitive assessment of structural capability has also increased although survey procedures remain much the same as for smaller vessels—with the physical capability of the Surveyor a severely limiting factor. Quality control in hull construction might well serve to remove or at least reduce arduous surveying times and such schemes are now being implemented in some large building ports. It is hoped this will result in common inspection schemes for new buildings in the future with much more information forthcoming on the influence of production standards on structural strength and reliability. Reliability, however, can only be quantified statistically and in this sense very little information is yet available. It is therefore only possible in this paper to give a broad qualitative picture of the existing situation regarding the influence of production techniques and material capability on product reliability.

2. SOME DESIGN CONSIDERATIONS

With increasing application of higher tensile steel in ship structures, nominal stress levels have risen. From the Rule formula the maximum yield stress corresponding to a K factor of 0·725 is 34·5 kg/mm^2 and consequently steels having a greater yield point than this cannot be employed to their full advantage. Quite apart from the nominal stress level, use of such steels should also be measured against the material and welding quality, workmanship and risk of fatigue and brittle fracture.

The whole presentation of the Rule treatment of ship's longitudinal strength has completely changed in recent years. The resulting increase in nominal stress levels has been small and can be justified by sound theoretical treatment of the subject (ref 3). The increase in transverse stress has been perhaps more noticeable. Consider, for instance, the bottom transverse of a large tanker and the flange stress induced in the bottom plating. The stress runs are perpendicular to the longitudinals and could perhaps cause buckling problems. A similar condition exists in way of large vertical webs on bulkheads. However, provided loadings are realistically assessed plate buckling should not be a problem. Similarly, as far as the Author is aware, the more pronounced bi-axial stress patterns which now exist have not resulted in any additional defects or practical problems.

* Principal Ship Surveyor, Gothenburg

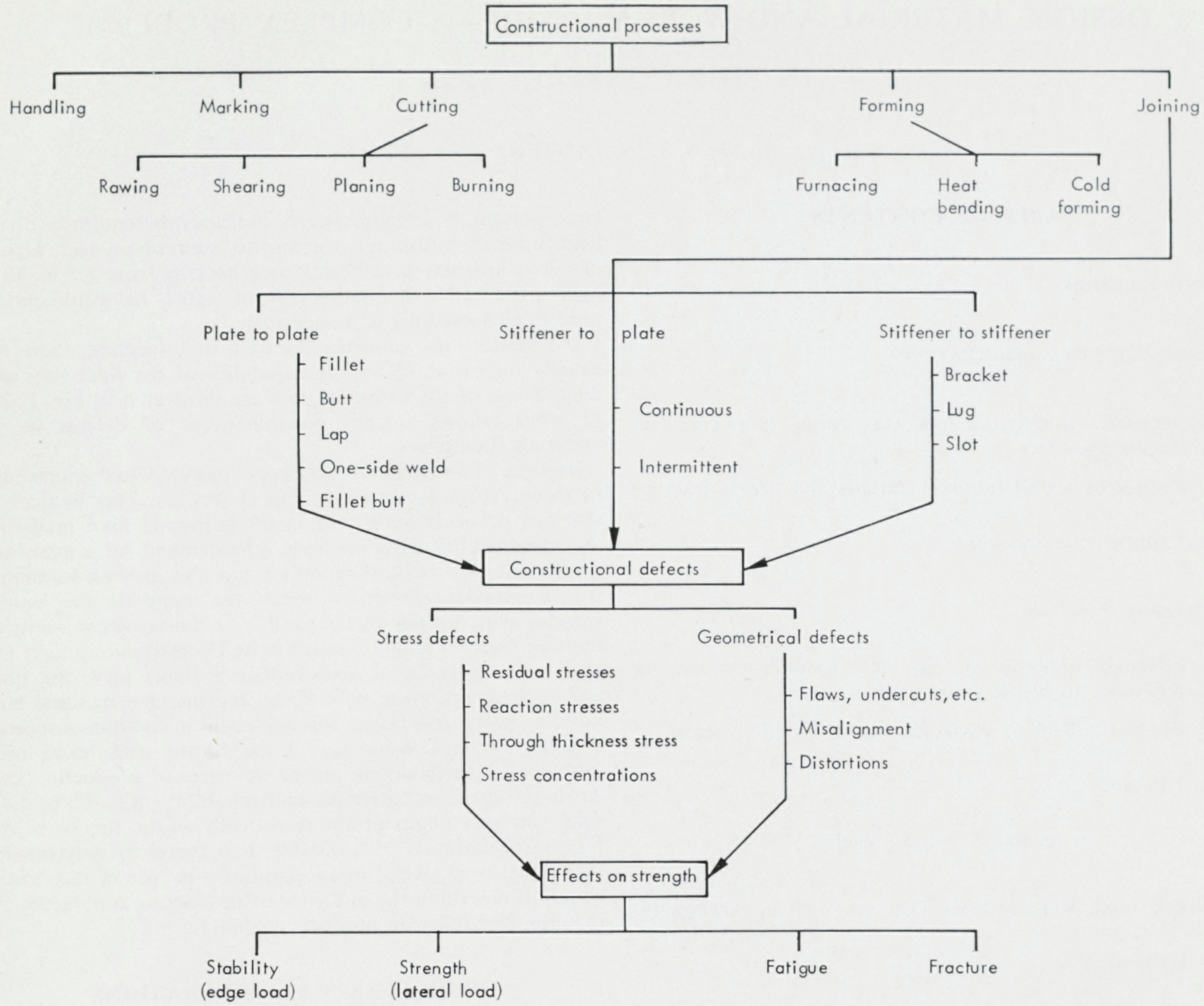


FIG. 1

Construction processes.

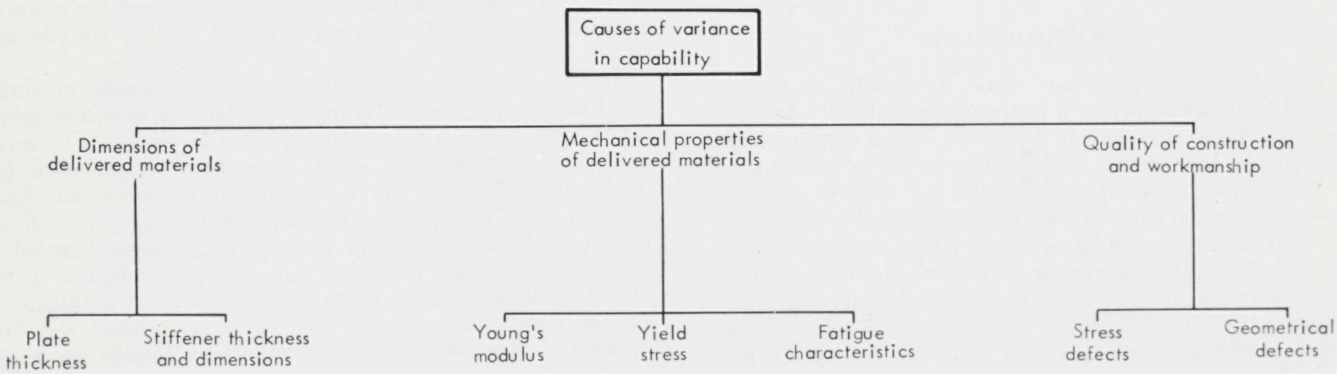


FIG. 2

Some possible defects in material quality.

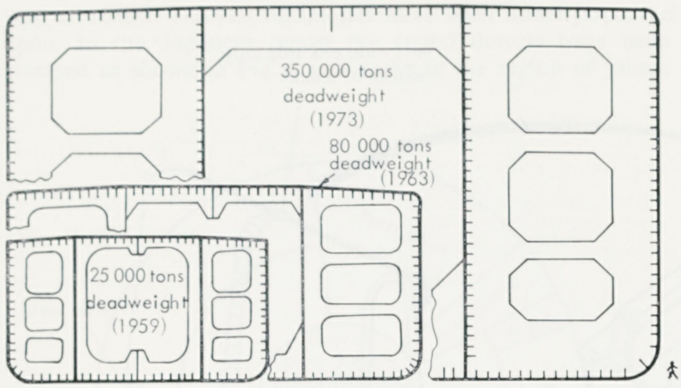


FIG. 3

Relative cargo tank sizes.

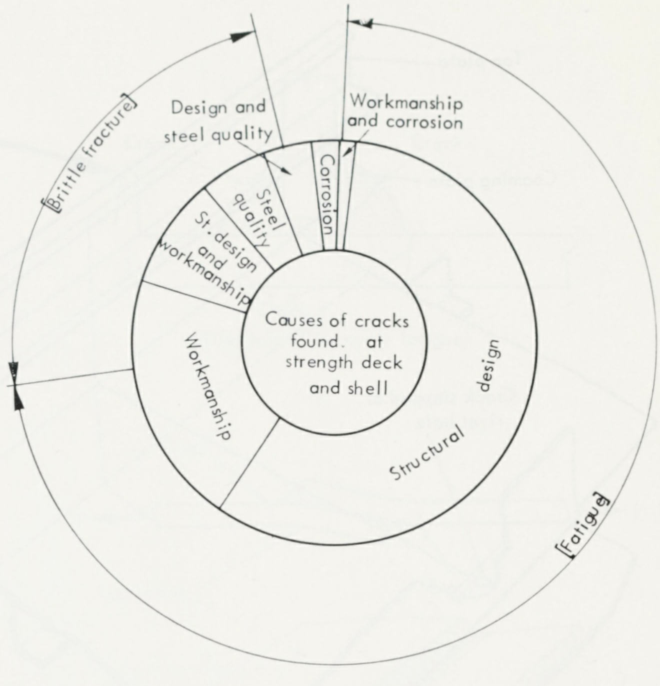


FIG. 4

Some causes of failures in shell and deck.

Recent designs have indicated a strong tendency towards a reduction in the number of primary support members in the ship. Whilst this may simplify construction and reduce building times it is liable to result in the fewer remaining members having larger deflections. These large deflections can influence stress patterns in secondary members and might give rise to additional problems in service. With more refined analytical tools these problems may be overcome, as shown later.

3. PERFORMED INVESTIGATIONS AND THEIR INFLUENCE ON EXISTING PRACTICE

3.1 Causes of actual failures: fatigue and brittle fractures

Before proceeding further, reference is made to an investigation of recorded defects in existing vessels. The results of this and other investigations have often been used, sometimes unjustifiably, as an incentive for the solution of fatigue and brittle fracture problems. Fig. 4 is based on the results from a Japanese investigation into the causes of failures to the shell and deck. The investigation embraced some 1200 vessels of various types which had been built over a 20-year period between 1950 and 1970 (ref. 4). In a sample of about 100 vessels, 144 cracks were discovered which were attributed to the causes shown in the inner circle. It can be seen that a major cause as concluded by the investigators was that of faulty structural design. The Author concurs in this conclusion which is felt would also apply if a similar investigation had been made at the same time on ships classed with the Society.

It would appear, therefore, that the hull designer and inevitably the plan approval Surveyor must accept some measure of the responsibility for the poor performance. In defence of this, it can be argued that many of the structural details at which the defects occurred, would be most unlikely to be submitted today and would certainly not be accepted for approval.

The nature of the failures was also established by the investigators and these have been summarised by the Author as indicated thus [ ] in the outer circle. It may be, however, that the more formal and scientific concept of fatigue is disguising failures which are due simply to physical underdimensioning and the enormous stress concentrations which result.

Two examples of brittle fractures which were illustrated in the Japanese report are shown in Fig. 5 and Fig. 6. The former shows a hatch coaming where the most severely stressed item had been designed such that a back run could not be made at welded joint. The crack started at the top and ran down through the coaming and the deck. The bilge keel shown in Fig. 6 has obvious discontinuities and concentrations of high stress. Such designs were probably common in the 1950 and 1960 period. Both, however, have questionable relevance to today's judgement of the risk of brittle fracture.

3.2 Failures at connections

Areas of internal members of cargo holds and tanks in large ships which were considered to be defect prone have also been investigated in Japan (*see* Fig. 7). A large number of failures were found at the connection of the bottom longitudinal to the bottom transverse and in way of bracket toes in the centre tank. In the tank corner connection, where high shear stresses exist, many buckling failures were recorded. This fairly familiar situation was the subject of an earlier paper by the Author and was also considered in a large Swedish investigation (ref. 5).

Upon closer examination, the investigations reveal that these particular types of failure occurred mainly on ships built before 1968, subsequent to which, design calculation

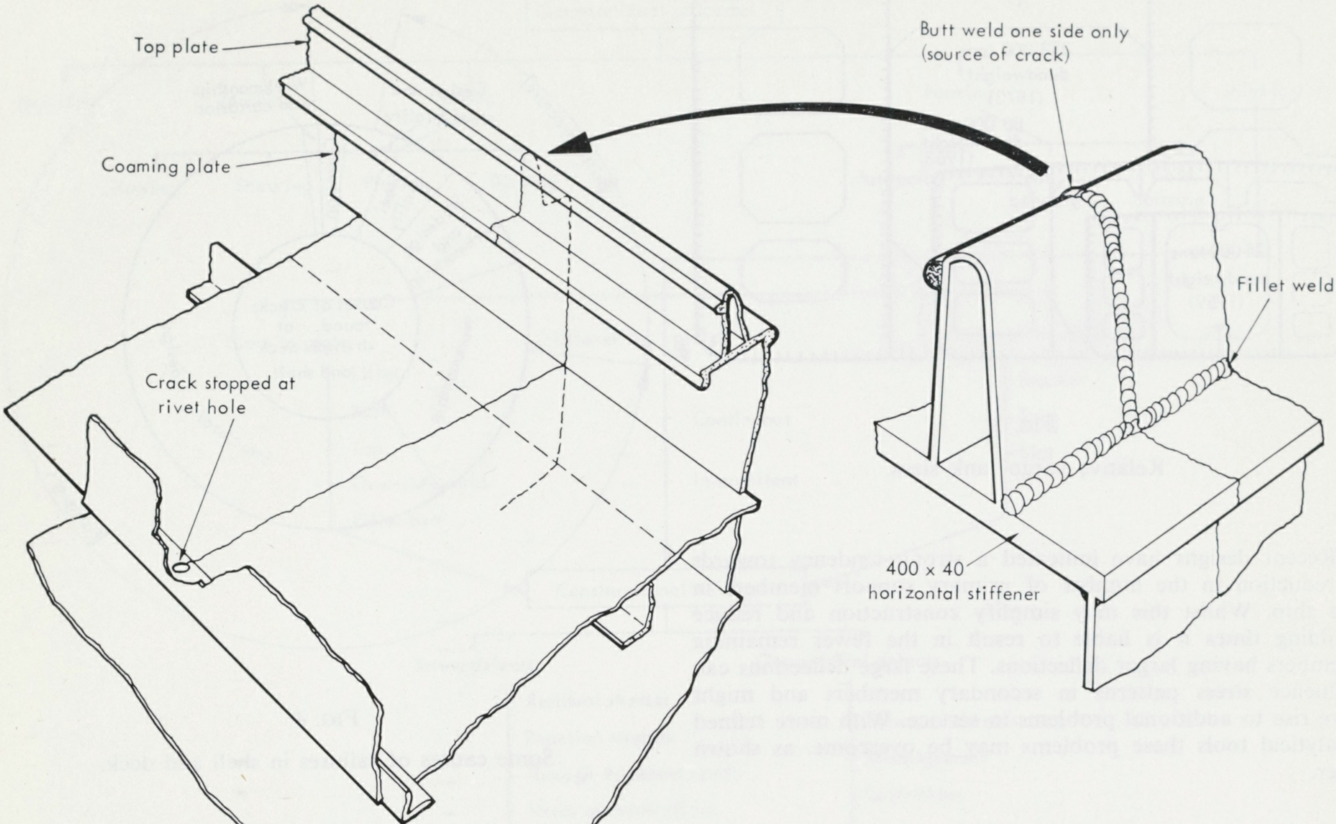


FIG. 5

Fracture in hatch coaming.

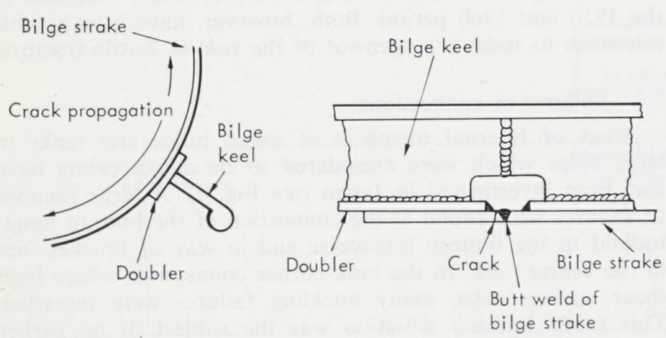
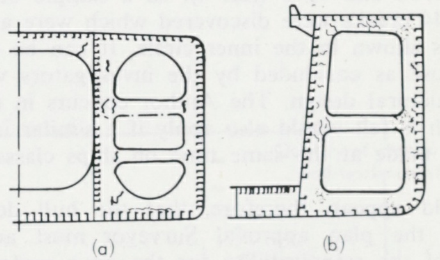


FIG. 6

Fracture at bilge.



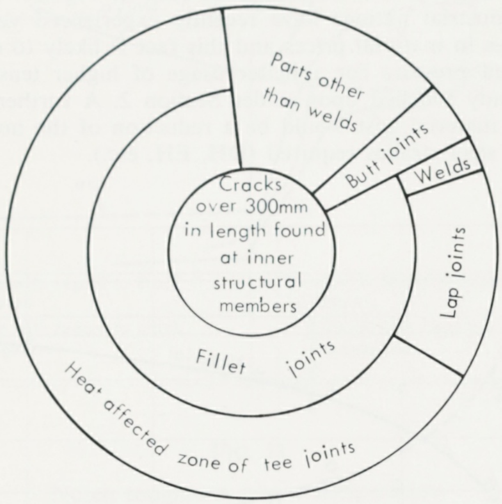
(a) Typical fractures at transverse internals of oil tankers

(b) Typical fractures at transverse internals of ore carriers

FIG. 7

Areas prone to damage.

and structural analysis techniques have been much improved upon. In the Japanese report the found defects have been grouped as shown in Fig. 8 and occur in the region of joints.



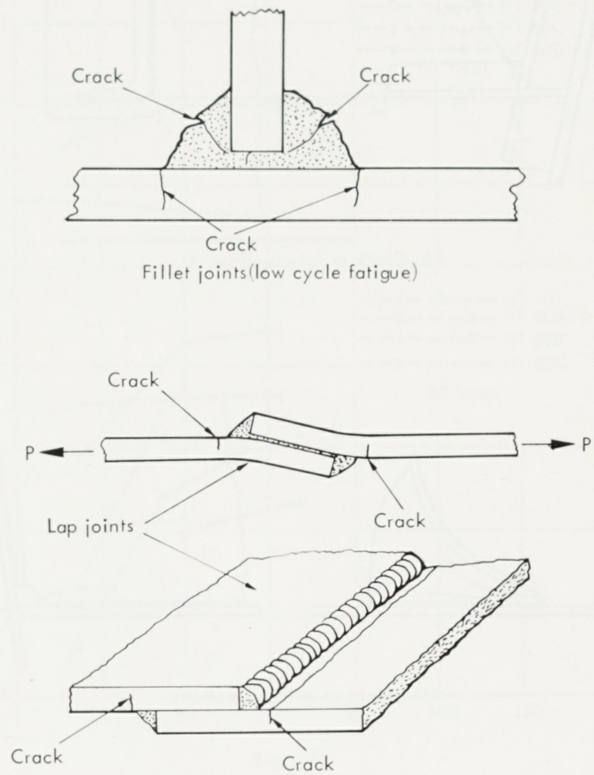
(b) Large cracks occured in internals of large ships (656 cracks)

FIG. 8

Some causes of failures in internals

These joints were made by welding and whilst it is clear that many were in areas of existing high stress concentration, it is interesting to note the distribution of failures according to the type of welded joint used. The large number of the fillet joint failures shown in the illustration merely indicate the very common use of the tee-joint in ship construction. The lap joint shown in Fig. 9, which is less common in Scandinavia seems to have an inherent tendency to crack in the area shown, particularly in those structures subject to low cycle fatigue.

The remaining types of failures shown in Fig. 9 have, quite rightly, been attributed to high cycle fatigue and are mainly the examples of failure which can be found at the aft end of a vessel, although they are often present in other areas of the vessel such as at bracket toes and cut-outs for longitudinals, etc. Whilst all these failures occurred at welded joints very little blame can be attributed to the choice of electrode or steel used. The Author merely wishes to indicate that in most cases the failure was due to faulty detail design. Only a few examples have been shown and many more can be found in refs. 5 and 6.



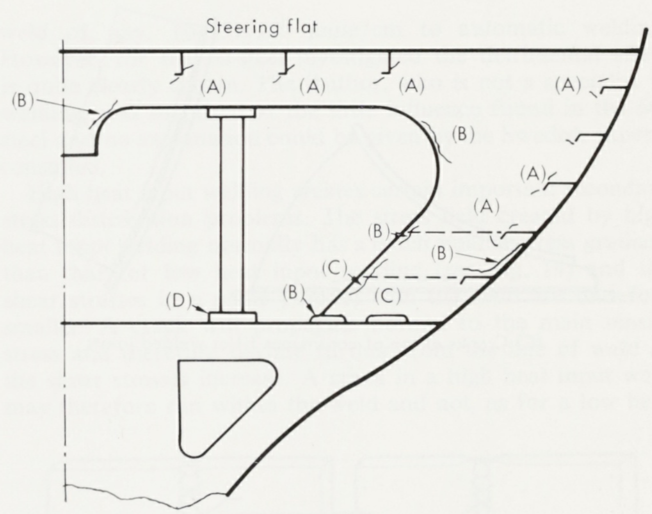
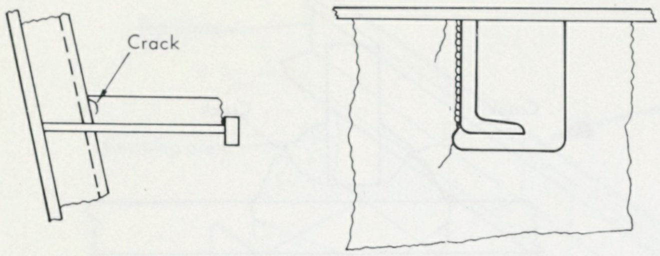
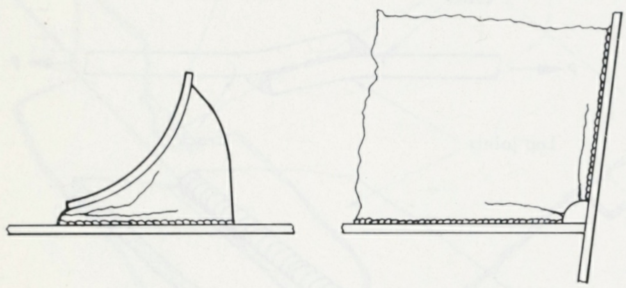


FIG. 9

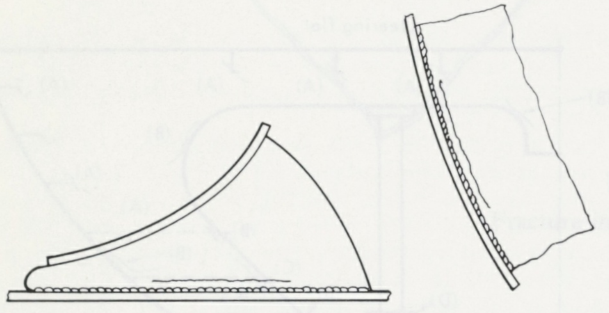
High cycle fatigue failures.



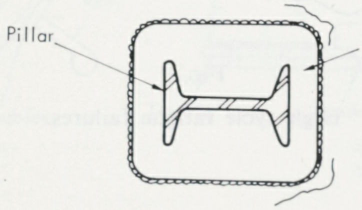
(A) Crack at intersections



(B) Cracks at end connections



(C) Cracks at toe of continuous fillet welded joints



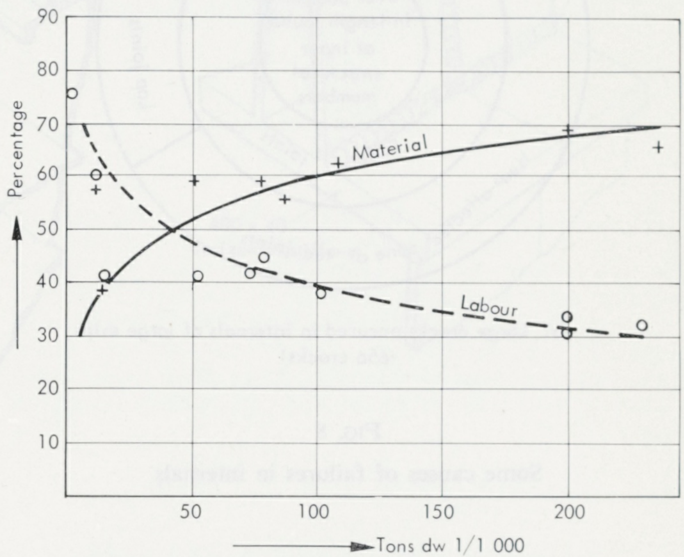
(D) Cracks around doubler

FIG. 9 (*continued*)

4. PRESENT POSITION

4.1 Effect of material quality and welding process on resistance to brittle fracture

With increased size of ship the cost of material can become larger than the cost of labour as seen in Fig. 10. (Ref. 1.) The industrial nations have recently experienced very large increases in material prices and this fact is likely to cause an increased pressure for a better usage of higher tensile steel, as already touched upon under Section 2. A further way to reduce material cost would be a reduction of the number of special steel strakes required (DH, EH, etc.).



MATERIAL AND LABOUR COSTS AS PERCENTAGES OF STEEL HULL COST

FIG. 10

As an illustration of some of the basic problems regarding material and fatigue, the Author has chosen some of the figures used in ref 7. This reference is a very good summary, and, although written some years ago, in the main it is still valid. Some of the basic concepts regarding material are illustrated in Fig. 11. The important point is the notch toughness curve, shown marked 'kV'. Two temperatures are stated; to the right of T_c the material is tough with regard to crack initiation. Such material is considered safe with regard to initiation and is what is practically aimed for. Obviously it would be preferable to have material which is also tough with regard to propagation and that is achieved if the kV value lies to the right of T_S.

The notch toughness curves (Charpy curves) for some different types of steels are shown in Fig. 12. As a large investigation regarding notch toughness has been recently carried out in Sweden, the Author has summarised the results and also indicated these in the figure (heavy dotted lines).

As seen, the 36-steel has good notch toughness. A ship built today of Scandinavian higher tensile steel has, therefore, equal or better notch properties than an older ship, built entirely in D-quality, i.e. the material is safe against crack initiation. Where, due to production reasons, a weld of high heat input type may be made the properties are, as shown,

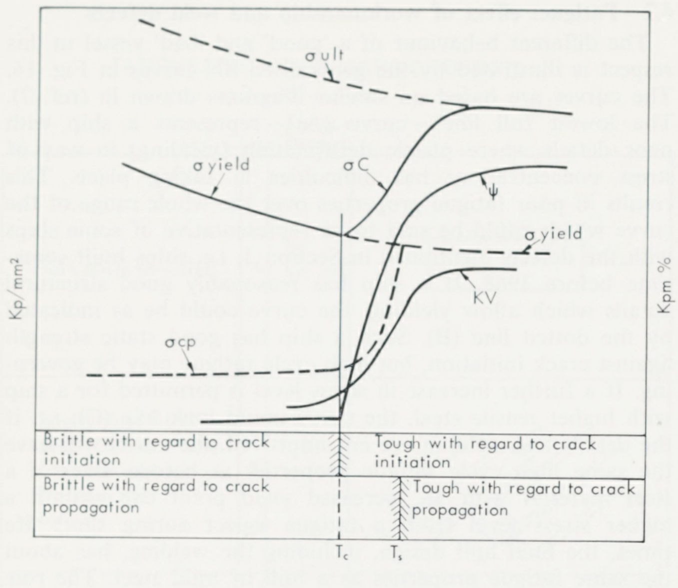


FIG. 11

Notch toughness *versus* temperature.

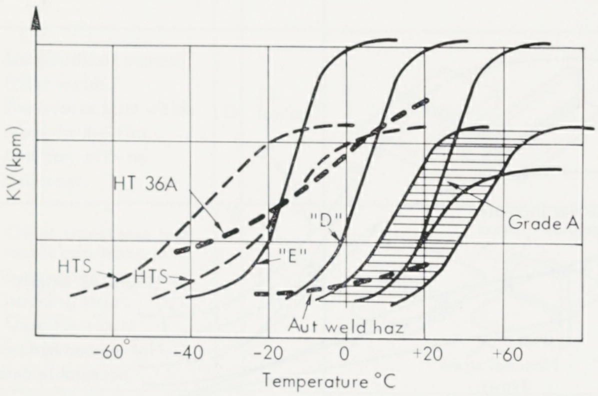


FIG. 12

Notch toughness curves for different steels.

not very encouraging. The notch toughness at the weld is much lower than that for the steel, the difference being greater if higher tensile steel is used. During normal production the kV value may, in many circumstances, be not more than say 2 kg m at 0°C. A butt in the rolled stiffener and a butt joint in the high heat input welded plate may often lie in the same line (with certain exceptions). A high heat input welded assembly should therefore be examined with care, although there is little real evidence of an increased number of brittle fractures resulting from this process.

Due to embrittlement of the steel material caused by high heat input welding, several investigations have been made. The Japanese investigation, shows in Fig. 13, that at least for the 60-steel there was no particular difference in the initiation temperature for brittle fracture when changing from a manual

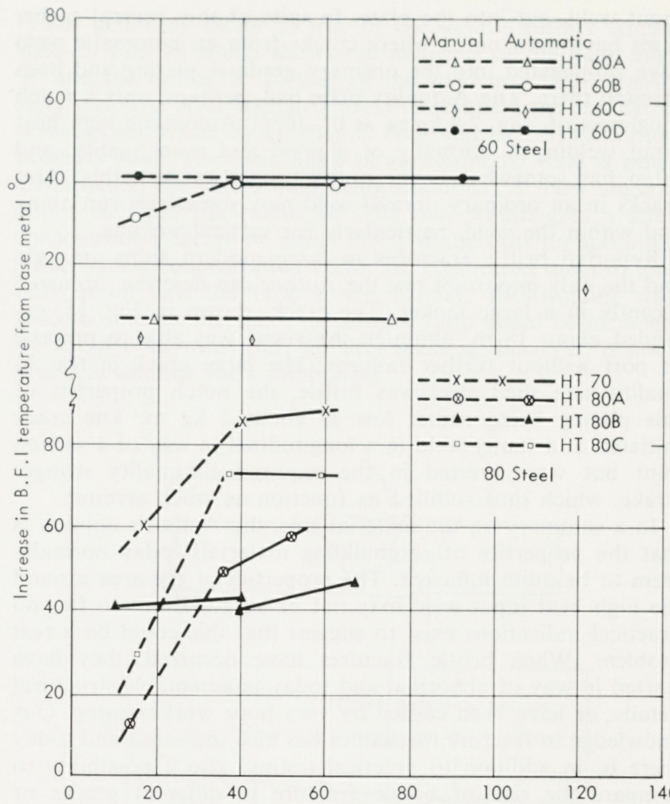
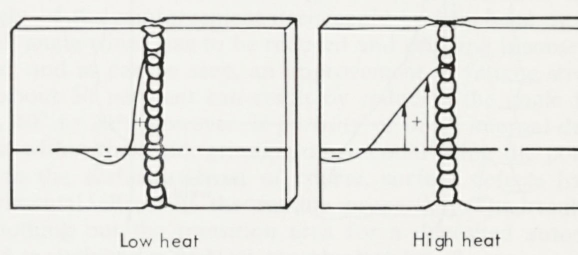


FIG. 13

Initiation temperatures for different steels.

weld of, say, 10–15$\times 10^3$ joule/cm to automatic welding. However, for the 80-steel investigated the detrimental effect is quite clearly shown. The Author, who is not a specialist in welding, was surprised at the little influence found in the 60-steel and no explanation could be given by the Swedish experts consulted.

High heat input welding creates certain important secondary stress distribution problems. The stress field created by high heat input welding normally has a much smaller stress gradient than that for low heat input welding (*see* Fig. 14) and the shear stresses in a plane parallel with the weld are therefore smaller. A crack will propagate normal to the main tensile stress and therefore deviate further from the line of weld as the shear stresses increase. A crack in a high heat input weld may therefore run within the weld and not, as for a low heat



STRESS GRADIENTS WITH LOW AND HIGH INPUT WELDING

FIG. 14

input weld, out into the plate. In spite of this, several earlier cases have been noted where cracks from an automatic weld have propagated into the ordinary grade A plating and been arrested there. The A-quality plate had, perhaps, only a notch toughness of, say, 2·8 kg m at 0°–10°C. Automatic high heat input welding is normally of a good and even quality, and often has somewhat better notch properties than this. Also cracks in an ordinary normal weld may sometimes run along and within the weld, particularly for vertical welding.

Reported brittle fractures in large modern ships are rare and the only important one the Author can describe, occurred recently in a large tanker. The crack, shown in Fig. 15, extended about 18 m, although the vessel was able to proceed to port without further failures. The large crack in the A-quality side shell steel was brittle, the notch properties of this plating being rather low at about 2 kg m. The crack initiated at a faulty weld in a longitudinal in way of a section joint but was arrested in the required Eh-quality stringer strake, which thus fulfilled its function as crack arrester.

In a summary on the material side, the Author's opinion is that the properties of shipbuilding materials today normally seem to be quite sufficient. The properties of the area around the high heat input weld may not be so good, but so far, no practical indications exist to suggest that this could be a real problem. When brittle fractures have occurred, they have started in way of abnormal and today unacceptable structural details, or have been caused by very poor workmanship. Our knowledge in fracture mechanics has also increased and today there is, in addition to practical testing, also a possibility to compare the risk of brittle fracture in different grades of material by using developed theoretical calculation models.

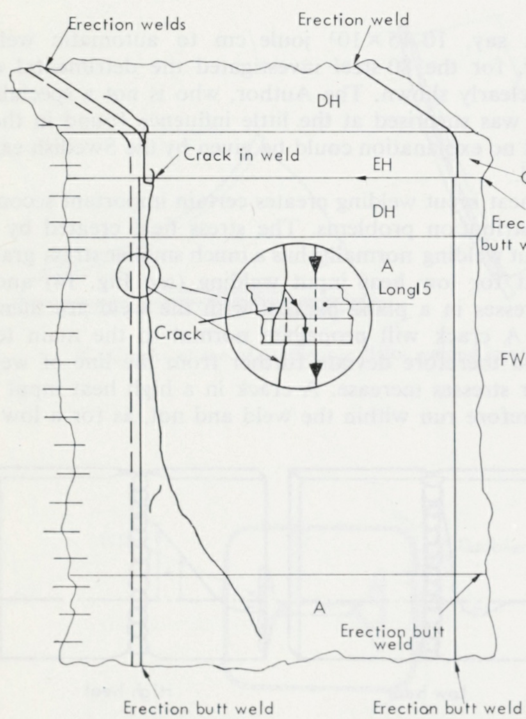


FIG. 15

Brittle fracture in side shell.

4.2 **Fatigue: effect of workmanship and weld defects**

The different behaviour of a 'good' and 'bad' vessel in this respect is illustrated by the generalised SN-curves in Fig. 16. The curves are based on similar diagrams drawn in (ref. 7). The lowest full line — curve (A) — represents a ship with poor details where plastic deformation (yielding) in way of stress concentrations has difficulties in taking place. This results in poor fatigue properties over the whole range of the curve which could be said to be representative of some ships with the defects mentioned in Section 3, i.e. ships built sometime before 1968. If a ship has reasonably good structural details which allow yielding, the curve could be as indicated by the dotted line (B). Such a ship has good static strength against crack initiation, but high cycle fatigue may be governing. If a further increase in stress level is permitted for a ship with higher tensile steel, the curve could look like (C), i.e. if the details and welding are not improved, the vessel will have the same high cycle fatigue properties as before. Even if a steel material with an increased yield point can sustain a higher stress level from a fatigue aspect during short life times, the final hull design, including the welding, has about the same fatigue properties as a hull of mild steel. The role of welding and the various types of joint are therefore of paramount importance, as can be adjudged from the findings illustrated in Figs. 4 and 8.

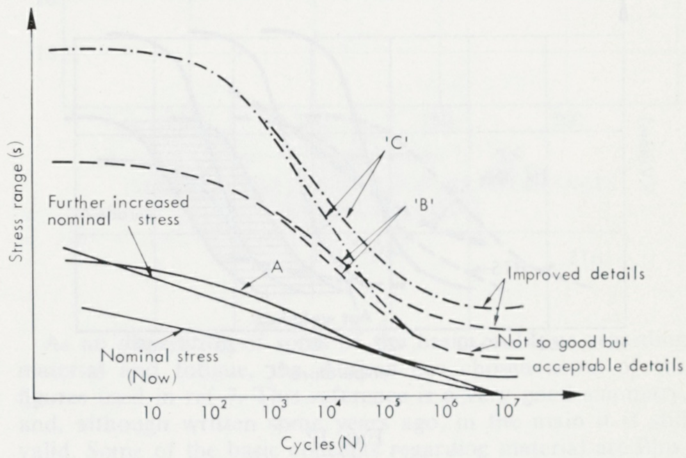


FIG. 16

Generalised SN curves.

On the solution of fatigue problems there are today many calculation programs available to estimate the limiting fatigue life (ref. 8). SN curves and estimated load history form the necessary basic information. The SN-curves which form the basis of the well known British Standard sheets shown in Fig. 17 can be used, the joint fatigue limits being as shown. As can be seen from the values for class B, C and D welds, there is a large difference between a manual and an automatic weld, as well as a longitudinal and a transverse weld. It is the stress direction and geometry which govern, not the properties of the deposited weld material in itself.

FATIGUE STRENGTH PULSATING TENSION

at 2×10^6 cycles

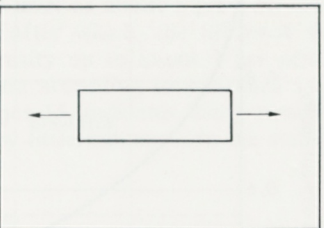
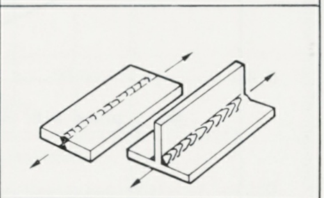
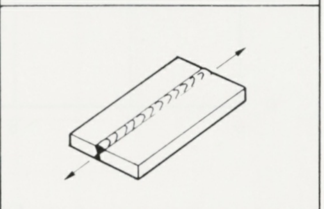
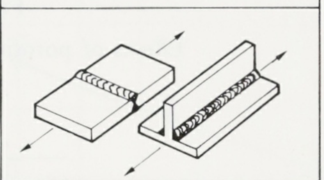
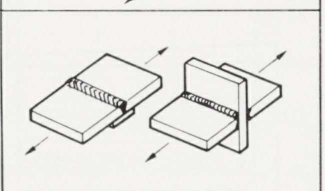
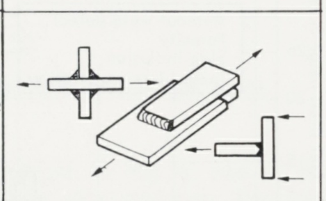
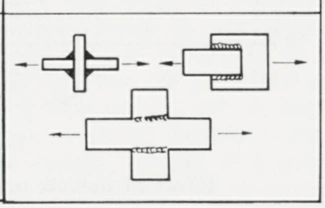
	Class	Fatigue strength	
Plain plate as rolled	A	25 kp/mm²	
Longitudinal fillet and butt welds made with automatic process. (No stop/start postions)	B	kp/mm²	
Longitudinal manual butt welds	C	16 kp/mm²	
Longitudinal manual fillet welds. Transverse butt welds made in the flat position with no undercut.	D	13 kp/mm²	
Other transverse butt welds and transverse butt welds made on backing strip. Cruciform butt welds.	E	11 kp/mm²	
T-butt welds. Transverse non- load carrying fillet or butt welds and weld ends Transverse load - carrying fillet welds of type shown.	F	7,5 kp/mm²	
Transverse or longitudinal load-carrying fillet welds Welds on or adjacent to plate edges.	G	6 kp/mm²	

FIG. 17

Joint fatigue limits (British Standard).

The geometry factor in way of the weld is further emphasised in Fig. 18 which shows the influence of the flank angle 'θ' and toe radius 'R' in way of the weld in a higher tensile steel (ref. 4). Also included are the results from an earlier investigation made by Gurney & Newman (ref. 9) on a more ordinary steel. The flank angle in an automatic weld is generally smaller, the whole weld being smoother. A flank angle of about 60° and toe radius of 0·02 mm would give a stress concentration factor of about 3·5. Fatigue tests on specimens made with a concentration factor of 3·5 showed them to have only about half the fatigue life. If, say, TIG-welding were used to smooth the weld, the fatigue limit could be increased by about 6 kg/mm² (ref. 4). TIG dressing, however, is not a practical solution for a shipbuilder.

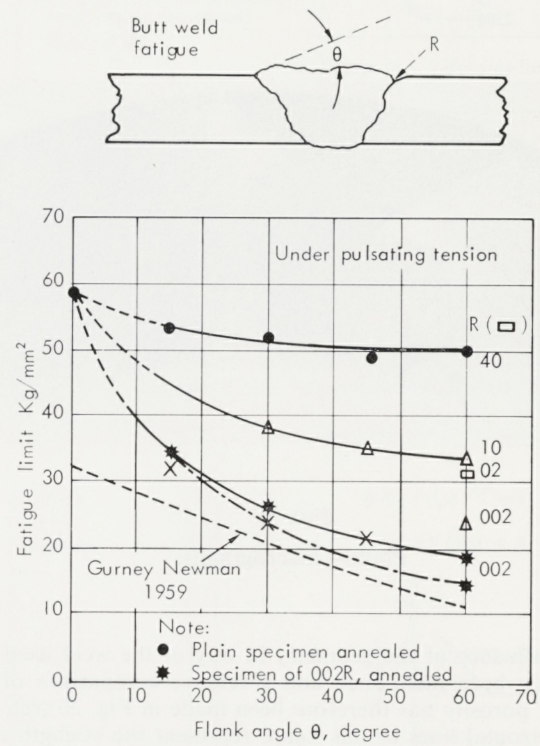


FIG. 18

Influence of weld geometry on fatigue limit.

A common misunderstanding is that a reduction in the height of the reinforcement itself could be beneficial; it is the flank angle which has to be reduced and grinding is sometimes used, and as can be seen, an improvement in fatigue strength of about 30 per cent can result by reducing the angle from, say, 40° to 20°. However, if porosity or other internal defects exist within the weld, grinding-down could bring the porosity up to the surface, where, of course, surface defects have a detrimental effect on the fatigue properties. The results of smoothing out the transition area for a deposited automatic weld material in a 36-steel parent plate by re-running with a 3·25 mm electrode were not satisfactory as can be seen in Fig. 19. The run made was sufficient to precipitate a crack and illustrates again that weld repairs should be done only when *absolutely necessary*.

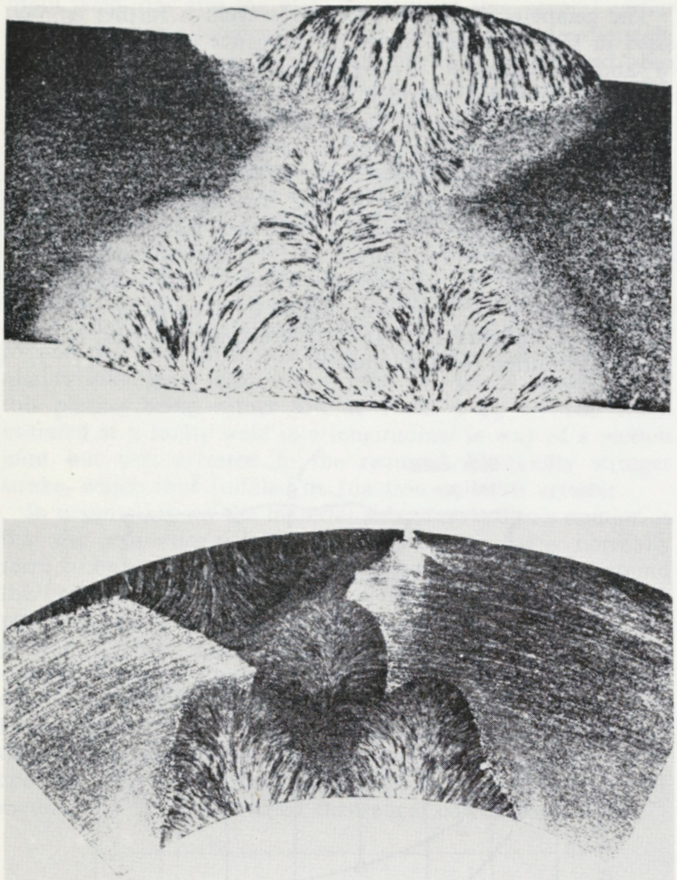


FIG. 19

Photo—macrographs.

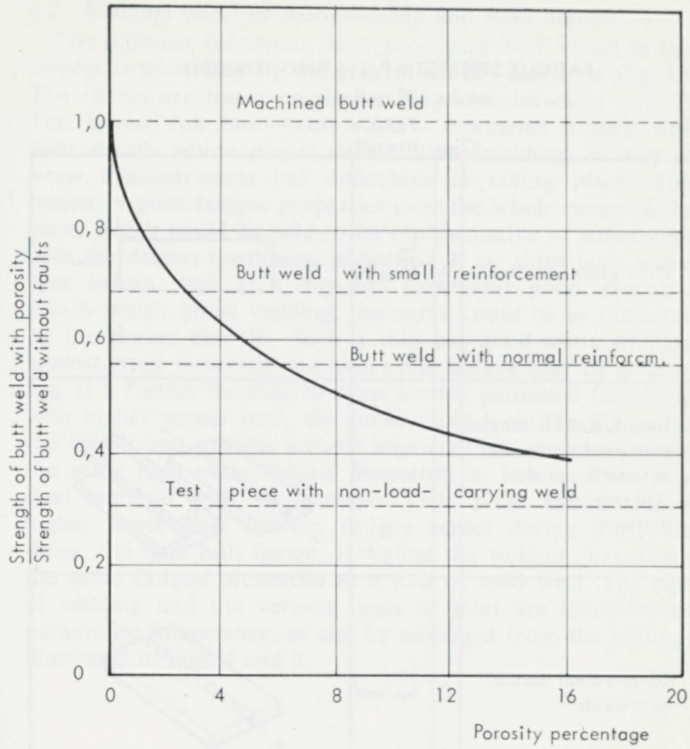


FIG. 20

Effects of porosity on weld strength.

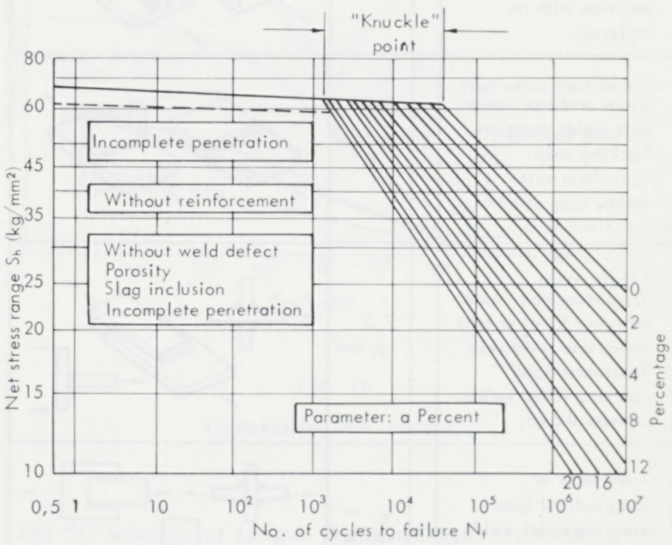


FIG. 21

Effect of defects on weld fatigue strength.

The influence of the geometry in way of the weld itself has previously been mentioned and a relative comparison of the effect of porosity has therefore been made in Fig. 20 (ref. 10). The horizontal lines in this figure represent the strength ratio between good butt welds and welds with different degrees of reinforcement and also non-load-carrying fillet welds (T-joint). The porosity curve and the line for a butt weld with normal reinforcement intersect at about 5 per cent porosity and normally therefore this amount of porosity will not further decrease the fatigue strength of the design. There is still a large safety margin even with higher porosities at the horizontal line representing a T-joint. With a level of up to 3 per cent volume there appears to be no practical risk whatsoever of reduced fatigue life (ref. 1).

In reference to porosity, etc., it is worth mentioning, that a large series of test specimens including a large test section of about 100 tons have been subject to shock loads at different temperatures and the effects investigated for these types of faults, i.e. porosity, slag inclusion and lack of penetration (ref. 10). These defects were found to have only a small effect on the results obtained where the welding had been performed with basic electrodes. With rutile electrodes, however, very severe cracking occurred for small defects subject to shock loading.

Porosity in welds may be easily seen with non-destructive testing. The severity of faults such as porosity, slag inclusion, incomplete penetration, etc., are often expressed as percentage area of defects per total weld area. Typical general fatigue curves for these defects are shown in Fig. 21 (ref. 12) and of particular interest is the knuckle point, i.e. the point of

inflection. It could be mentioned here that in many other fatigue investigations on the effects of geometrical defects, such as weld reinforcements, misalignments, etc. (ref. 13), the knuckle point in the fatigue curve is situated around 10^4 reversals. Fig. 22 (ref. 12) illustrates this point and it can be seen that up to a defect severity level of about 5 per cent the knuckle point is much affected after which, the influence is smaller. For static strength, porosity up to about 7 per cent could be a suitable limiting figure according to reference 14. Regarding slag inclusion, reference 15, indicates that porosity up to about 10 per cent has very little influence on the static strength.

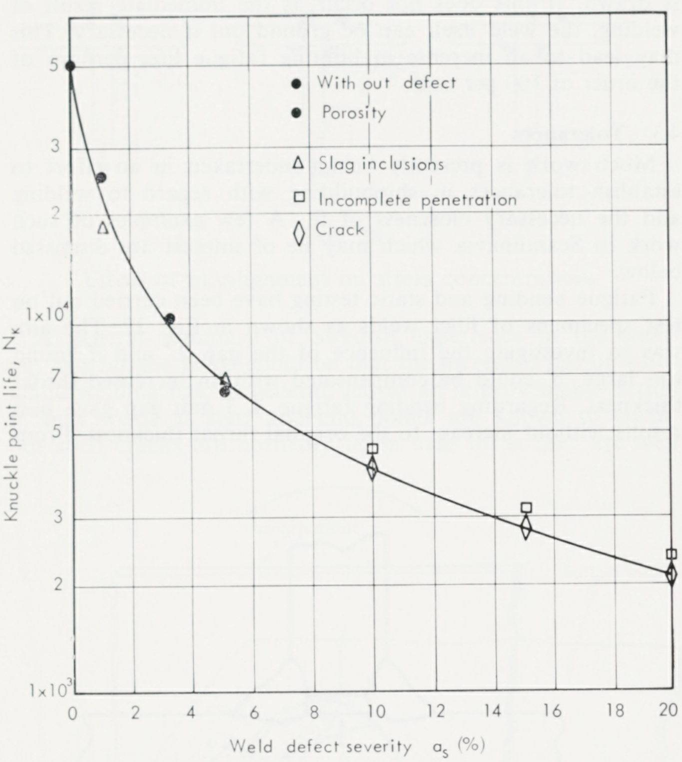


FIG. 22

Influence of defect severity level on knuckle point.

Fillet welds are, as already shown in Fig. 8, one of the main source points for cracks in a ship structure. The fillet weld is also one of the most common welds in shipbuilding and the general behaviour of this weld is often used as a reference point. Several investigations into the properties of this type of weld have therefore been carried out. A load-carrying cruciform fillet weld has, as shown already in Fig. 16, only about one-third of the fatigue strength of a good butt weld, but this can be used as an argument for accepting, for instance, a large percentage of porosity as intimated in connection with previous figure. However, the fatigue strength of a fillet weld can be increased as shown in Fig. 23 (ref. 4). Comparing the fatigue strength of type A (full penetration) with type C (half penetration) it is seen that the fatigue limit is only about half of that of a fully penetrated joint, in this case 7 kg/mm² and 15 kg/mm² respectively. These figures refer to testing in air. Corrosion fatigue tests gave results about 50 per cent of those obtained in air.

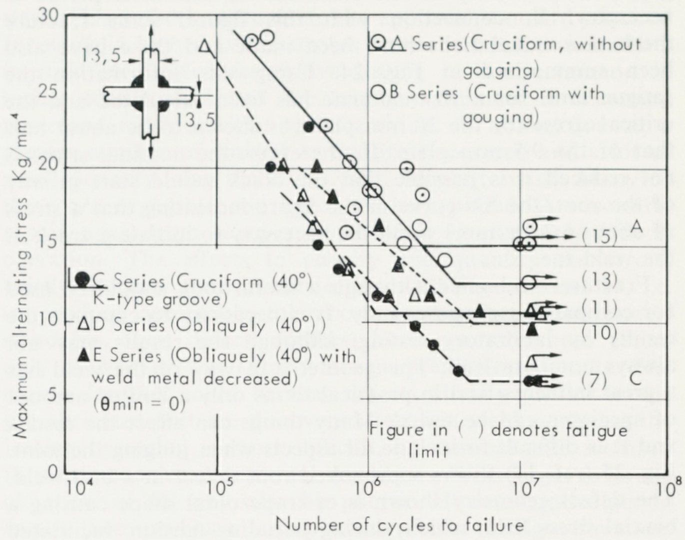


FIG. 23

Strength of fillet welds.

The fillet weld, subject to tension fatigue in Fig. 24, can fail either at the toe or in way of the root. Of the specimen shown about equal proportions cracked at the weld toe and the weld root (ref. 16). The practical experience of many Surveyors

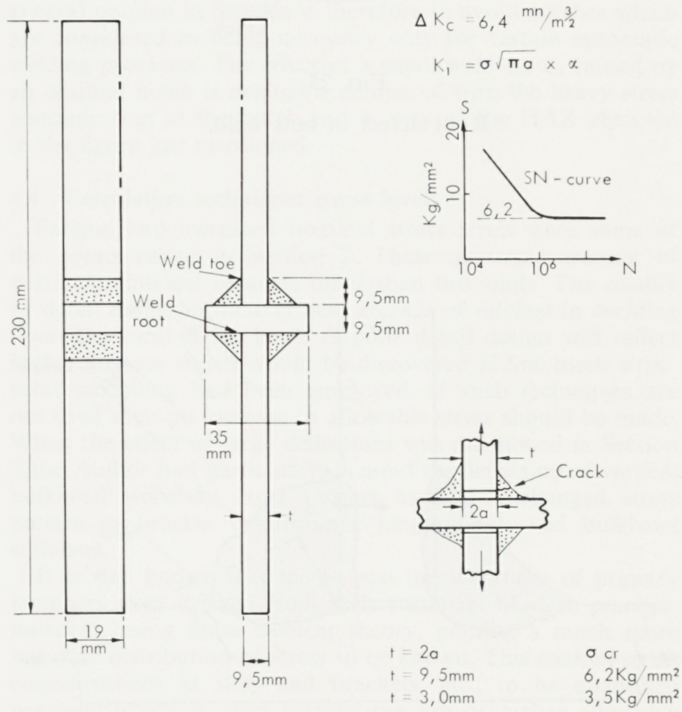


FIG. 24

Cracks in fillet weld subjected to tension fatigue.

would support this fact. A crack at the toe of a sound weld requires a certain initiation time, whilst at the root there is a gap (crack) already at the start, i.e. no initiation period is necessary. In connection with this figure, some fracture mechanics calculations have been made and these have also been summarised in Fig. 24. Using this information the fatigue limit for a 30 mm plate has been calculated and the critical stress for the 30 mm plates is shown to be about half that of the 9·5 mm plate. If, therefore, the nominal stress is not reduced it is possible that the crack would start in way of the root, the SN-curve in the figure indicating that a stress of about 6·2 kg/mm² would be necessary to initiate a crack at the weld toe.

Fracture mechanics, although a useful tool, should be used for comparison purposes only. It is necessary to compare the results by laboratory testing, although the results may not always compare well. The geometry in way of the weld has a great influence and in practical terms only a limited amount of specimen can be tested. Many things can affect the results and it is difficult to include all aspects when judging the joint. Fig. 25 (ref. 17) shows a provoked root defect in a butt weld. The defect geometry shown is of trapezoidal shape causing a biaxial stress field, which, during pulsating tension, facilitated the initiation of the fatigue cracks at the shorter side of the trapezoid. Root errors of a parallel epipedic shape were much less prone to cracking.

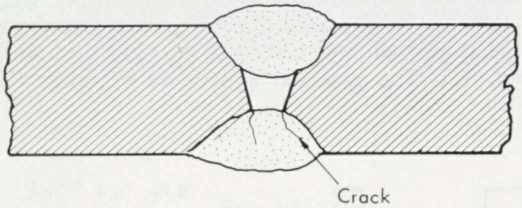


FIG. 25

Root defect in butt weld.

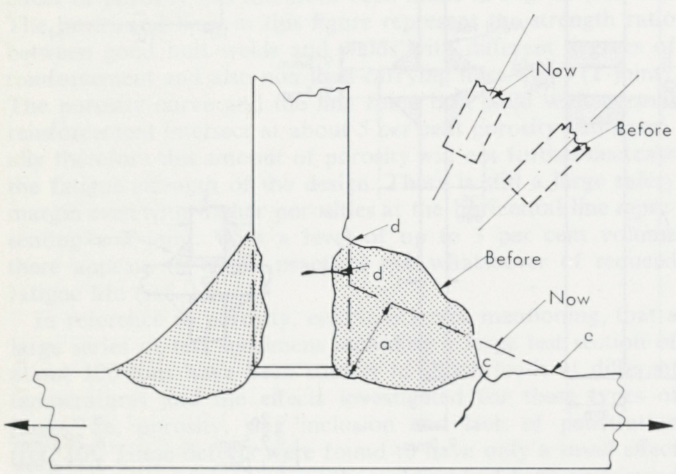


FIG. 26

Change in shape of fillet welds.

The Author referred to his outdoor colleagues in order to briefly summarise the change in fillet welds during recent years particularly with regard to weld geometry. The change in shape has been indicated by the dotted line on the right side of Fig. 26, which shows a non-load-carrying fillet weld. The weld has flattened out and in consequence there is likely to be a smaller stress concentration in way of the weld toe 'c'. This is, of course, an improvement provided the remaining throat thickness 'a' is checked. At 'd' the stress concentration has probably increased although this is of less importance in the type of loading shown. The positions and sizes of electrodes have also been indicated to give some explanation to the change. On the left side a concave, rather ideal, variation is drawn. If this does not occur as the immediate result of welding, the weld itself can be ground out if necessary. This may lead to an increase in limiting fatigue life, perhaps of the order of 100 per cent.

4.3 **Tolerances**

Much work is presently being undertaken in an effort to establish tolerances in shipbuilding with regard to welding and the necessary closeness of fit. A few examples of such work in Scandinavia which may be of interest are discussed below.

Fatigue bending and static testing have been carried out on test specimens of fillet welds as shown in Fig. 27. The aim was to investigate the influence of the gap 'b' and if found too large 'b' could be compensated with an increased throat thickness. Regarding bending fatigue, a 3 mm gap gave best results without increase to the original throat thickness. From

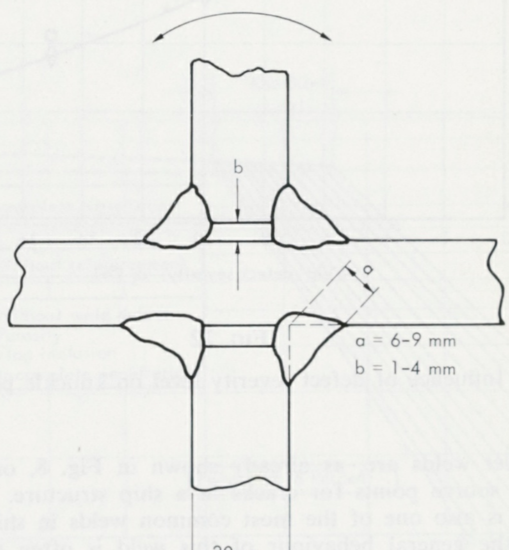


FIG. 27

Fitting tolerance: fillet weld test specimen.

static tensile tests a stress concentration factor (SCF) with a 1 mm gap was found to be of the order of 3·0 in way of the weld toe, and 3·2 in way of the weld root. Corresponding figures for a 4·5 mm gap were 2·2 and 2·4 respectively. In bending the SCF was reduced and was found for a 1 mm gap to be 1·6 in way of the weld toe and 1·2 in way of the weld root. Corresponding figures for a 4·5 mm gap were 1·5

and 0·5 respectively. An increased gap lowered the stress concentration factors. The effect of misalignment on the stress concentration factor can partly be seen in Fig. 28, where a misalignment from a half to the full plate thickness increased the SCF from 1·6 to 2·1 times that of a joint without misalignment. In the proposed Swedish Standard a misalignment of 0·2 T is considered an acceptable limit.

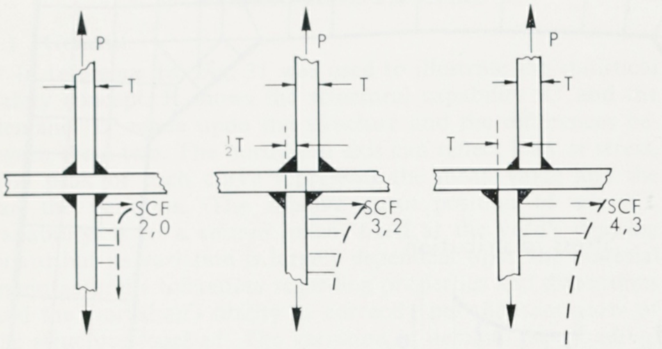


FIG. 28

Effect of misalignment on stress concentrations.

A general summary of Swedish experience in the welding and fatigue fields could be that the increased use of automatic welding has, in general, led to an improvement. Errors in butt welds at start and end points are known to occur, but such cracks are normally taken care of as they are well known and perhaps expected. The yards' records with regard to faulty joints generally show an increase in the number of errors when using higher tensile steel as compared with mild steel. This is fairly natural as the approval standard required for work in higher tensile steel is normally 4 compared with 3 in mild steel. Non-destructive testing is more common in higher tensile steel and the testing equipment is becoming more and more sensitive and searching. As a consequence, a greater number of smaller errors are discovered and a lot of small porosity, slag inclusions, etc., shown to exist. Many of these are negligible and need not be repaired. A successful repair, apart from being costly, is often an extremely difficult operation. The efforts to classify and quantify the errors, briefly touched upon in connection with Figs. 27 and 28, i.e. to produce acceptable tolerances, is therefore quite understandable. Real defects (cracks) have generally been found only in way of the joints of large prefabricated units. The reason for this could be that defects already present in the prefabricated unit are discovered and that welding during the joining operation is more difficult, i.e. vertical and overhead in rather poor environmental conditions. With larger units the closeness of fit may also be poorer. However, attempts to rectify these problems largely seem to be in the right direction. Considering the Author's earlier remarks about the properties of a transverse fillet weld and the generally good experience of the earlier type of transverse framing in way of deck and bottom, it is difficult to visualise any additional problems in today's longitudinally framed structures.

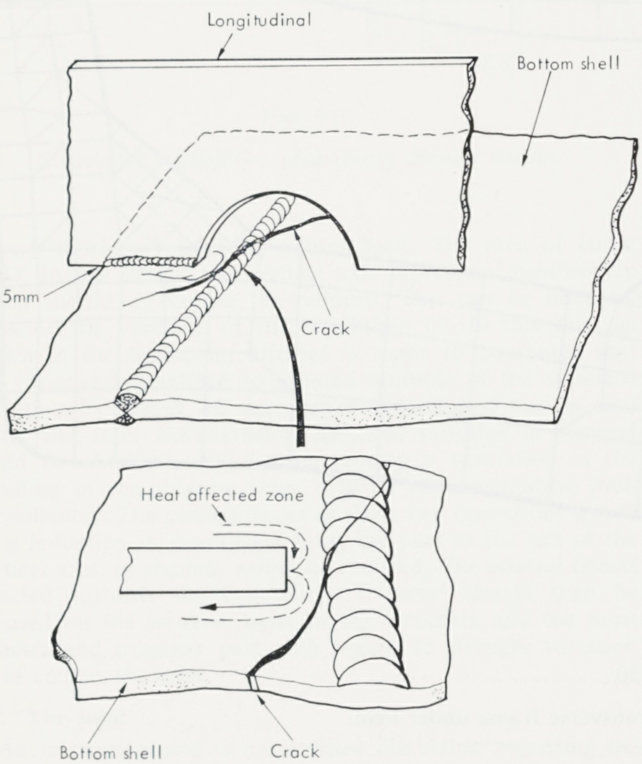


FIG. 29

Crack in shell bottom.

When the butt weld cracked as shown in Fig. 29 in way of a shell bottom A-quality steel plate in a modern semi-container ship, it was caused by the presence of the notch and its position. The cause could not be attributed to the weld or material. The notch ought not to have been in that location and in many cases such notches are frequently misaligned. The general opinion in Sweden is therefore to avoid notches which are considered as being necessary only for certain automatic welding processes. The effect of a small weld error, caused by an omitted notch is negligible compared with the heavy stress concentration at the notch end in way of the HAZ obtained in the figure just mentioned.

4.4 Calculation technique: stress levels

Fatigue and increased nominal stress levels were some of the points raised in Section 2. These questions are not of particular interest in areas other than the joints. The quality of detail design in these critical areas is of interest in deciding upon the local stress level. A poor detail design will reflect higher stresses which would be discovered if fine mesh structural modelling had been employed. If such techniques are not used then no increase in allowable stress should be made. When the effect of large deflections was mentioned in Section 2 the Author had particularly in mind the deflection of vertical bulkhead webs on large tankers causing a changed stress pattern in bracket connections, longitudinals and bulkhead stiffeners.

It is well known that in the past the scantlings of primary members were derived from Rule formulæ. Modern practice, however, using finite element theory, permits a much more accurate distribution of stress to be known. This enables stress concentrations at slots and brackets, etc., to be quantified precisely which in turn permits the use of higher allowable stress levels. Fig. 30 shows a transverse ring frame which under load has deformed to the shape shown. The lightly shaded areas are those where stress concentrations are very

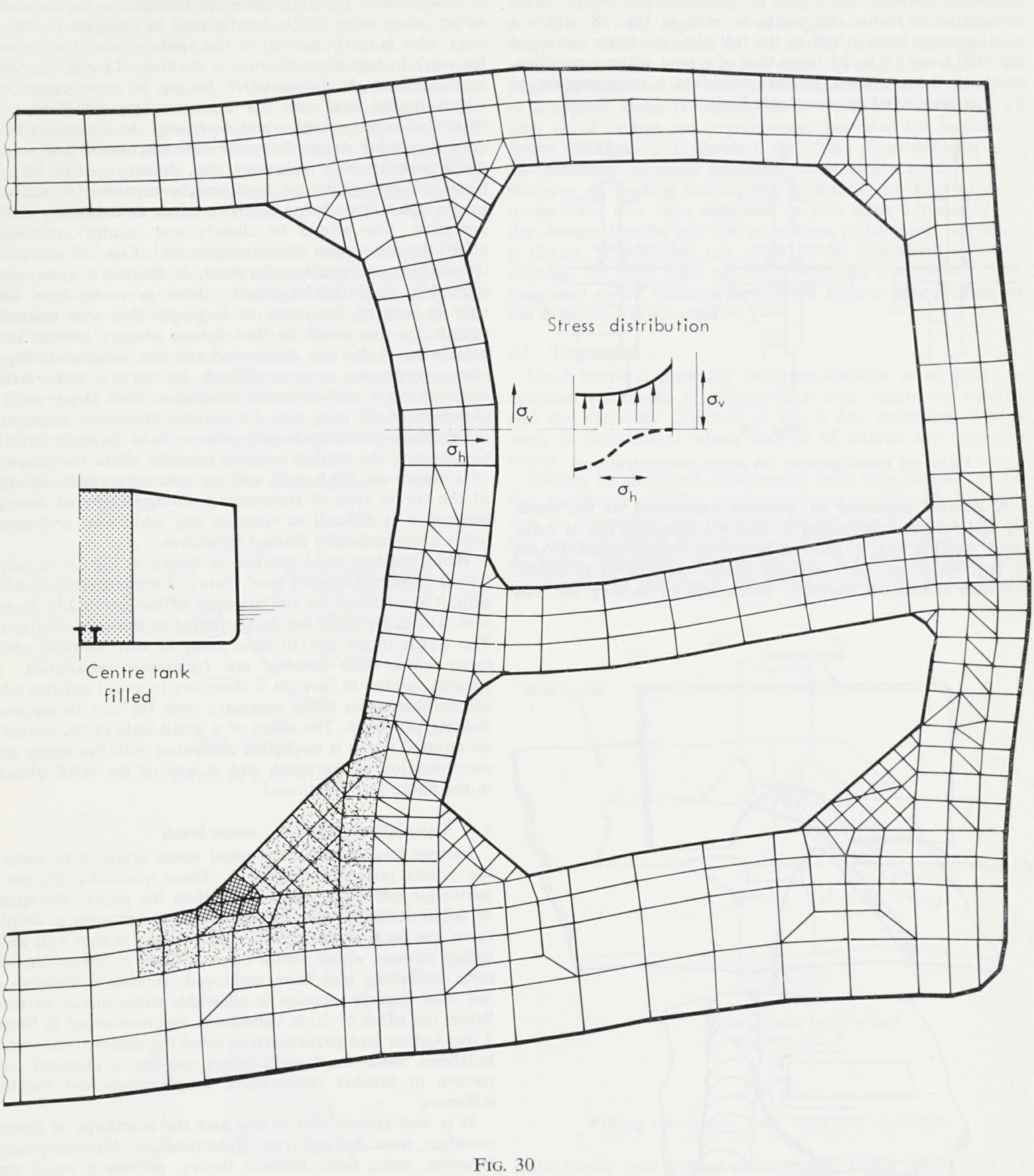


FIG. 30

Exaggerated deformation of transverse frame under load.

likely to exist and a finer analysis may be required. The darker shaded areas are those where the stresses involved were high enough to warrant an extremely fine mesh analysis. Ultimately it is possible to reduce the mesh size until only the weld itself is scrutinised.

5. STRUCTURAL RELIABILITY—THE STATISTICAL APPROACH

5.1 General

In reference 18, Fig. 31 was used to illustrate the statistical safety concept. It shows the structural capability 'C' and the demand 'D' made upon the structure and the differences between these two. The horizontal axis can reflect load or stress. The peak of each curve represents the mean value and the tail the variation. The relative mean position of say 'C', capability, is to a certain extent fixed at the yard's drawing board but its variation is largely dependent upon the material manufacturer's tolerances regarding properties and dimensions and the workshop's ability to correctly put different parts of the structure together. The variation in demand (say loading)

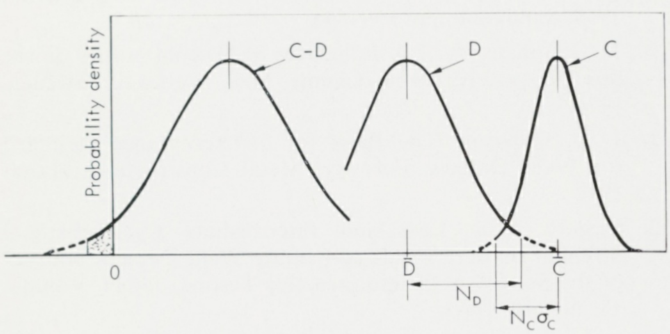


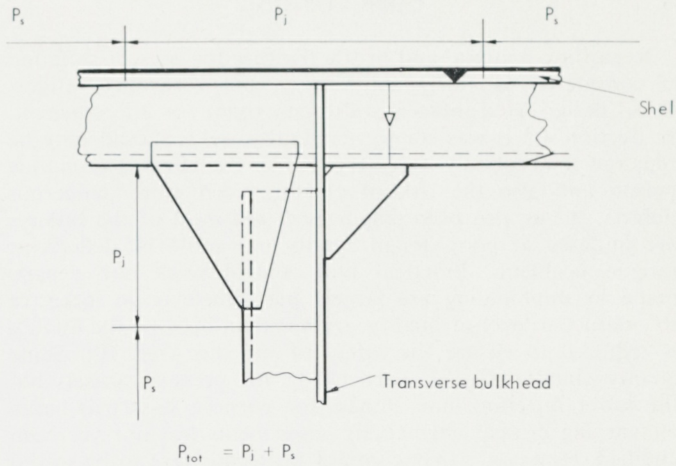
FIG. 31

Structural reliability: probability density curves.

is very much governed by nature itself. The area of curve 'C–D' to the left of the vertical axis represents insufficiency which should, of course, be reduced. This can be done by reducing the variance of the capability 'C'. In this case, as shown in the figure, an attempt is made to guarantee that the variance is confined to a small multiple of the standard deviation by means of, say, better control and testing, and better and safer workmanship. Reduced variation in demand could be obtained, say, by introducing a restriction in the handling of the ship by some type of instrumentation (hull surveillance). The consequences of these two operations would be a reduction in insufficiency, i.e. the part to the left of the vertical axis, as shaded, would be reduced. The control efforts directed towards the capability, 'C-curve' should then be focused on the weakest parts of the structure and the most difficult and irregular part with regard to strength variation is, of course, the joint.

5.2 The joint

An interesting and more detailed discussion regarding the conditions in way of the joint was also presented in ref. 18 from which the table in Fig. 32 is taken. It summarizes the quite different conditions outside and within the joint area.



	Joints	Structure clear of joint
Steel weight	Low	High
Variance in strength	High	Low
Labour cost per ton	High	Low
Material cost per ton	Low	High

FIG. 32

Failure probabilities in way of end connections.

Although the joint represents only a small amount of material, it involves much work, is often difficult to make and requires good workmanship. As the total probability of failure 'P' is the sum of probabilities of failures within 'P_j' and clear of 'P_s', it is of particular importance to control the joint with its greater variance in strength. The reliability of the joint is of course improved with a reduction in local permissible stress levels (previously discussed in connection with Fig. 29) or by a relocation of the joint in a low stress area. This latter is often impossible as can be seen in Fig. 30, where the brackets and joints could hardly be moved away from the highly stressed support point formed by bulkhead and shell. In addition to increased control and testing of the joint it is often possible to improve the situation by reducing the number of joints, particularly if this is requested by a production department.

In order to see if this could be practically done, two differing designs of a transverse ring of two 350 000 dwt tankers of the type illustrated in Fig. 30, building in different countries were compared and the following differences were found.

(a) The area of plate and stiffeners in one transverse ring on one vessel was about 20–30 per cent larger than on the other, i.e. 20–30 per cent larger area is exposed to corrosion.

(b) The number of stiffener ends was about two and a half times greater.

(c) The total length of stiffeners was about 30–40 per cent more on one with a corresponding increase in the number of fillet welds which must be laid.

6. CONCLUSION

Regarding material and brittle fracture the present situation is considered as fairly satisfactory, but regarding fatigue, detail design, etc., there is still much room for improvement. In Section 4.2 it was stated that faulty welds should only be repaired if absolutely necessary due to the obvious economic reason and also the risk of creating even more dangerous defects. It has also been emphasised that most of the failures are initiated at poor design details and could be defined as fatigue problems. Practical hull control work and repairs made in shipbuilding are largely performed as an incentive to produce a level of quality—rather than because the quality is required to ensure the safety of the ship (ref. 19). Some quality standards exist, particularly for pressure vessels but the lower rejection limit (fitness for purpose standard) based on varying critical engineering assessments has not yet been codified. However, fairly complex problems have to be solved before arriving at such standards. Such standards largely depend on the actual stress levels and expected number of stress reversals in addition to the type of defect expected, i.e. whether it be two-dimensional or three-dimensional. A meaningful use of such standards of course requires the ship to be divided into various zones with different acceptance and quality control levels. These areas must be defined by the yard at the design stage and by the classification societies during plan approval. The latter bodies, in spite of their enormous progress on the structural analysis aspect of approval, mainly approach matters in a traditional way. Efforts to establish a tangible system for the approval of workmanship and detail design, etc., are sometimes less than fruitful. This is perhaps understandable as the necessary background and requirements for such systems is still missing. In addition to the previously mentioned sub-division of the ship into various quality areas and the provision of fitness for purpose standards, classification societies also lack the necessary general procedures to match structural details to the expected demand in that particular area.

The above tools must be available before a really efficient quality assurance scheme and a correct plan approval procedure can be performed. Safety may be regarded as a statistical concept as briefly examined in Section 5 and little useful data is at present available regarding the actual capability of the different areas within a ship. With the refined calculation methods now used, there is also a tendency towards a fully stressed design. The simplified quality area divisions proposed in ref. 18 could therefore be easily criticised. However, with the ever increasing demand for reliability at low cost, the Author is convinced that there will, in the not too distant future, be means made available to enable a more comprehensive plan approval procedure to be adopted, which will include quality control and performance related standards.

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Discussion

on

Mr. S. T. A. Janzén's Paper

DESIGN, MATERIAL AND WORKMANSHIP— A COMPLEX PROBLEM IN SHIP CONSTRUCTION

Paper No. 1. Session 1975-76

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Discussion on Mr. S. T. A. Janzén's Paper

DESIGN, MATERIAL AND WORKMANSHIP—A COMPLEX PROBLEM IN SHIP CONSTRUCTION

Mr. J. J. MATHEWSON

The Swedish shipbuildng industry is well known for its progressive and scientific approach to the design and construction of ships and it is therefore of considerable interest to have current trends in that country presented to us at first hand by Mr. Janzén.

I well know from my many dealings with our Swedish clients that it is essential for the Society's credibility to be seen to be participating in pushing forward our frontiers of knowledge and Mr. Janzén has devoted considerable effort to this task. I am pleased to see in Scandinavia that reliability as opposed to just minimum weight is now accepted as a primary objective.

I should like to begin first with those matters on which we are in agreement, i.e. structural reliability must be approached in the manner outlined in the paper and, yes, it is a complex problem. I would suggest even more complex than indicated by the Author.

If one refers to Fig. 31 for example, the statistical distribution of capability given by curve 'c' is only applicable at the time of construction. A true assessment of probability of failure must take account of the variation in capability in service as a result of corrosion, etc., as shown in Fig. D 1 (ref. 20). The accumulation of the necessary data to be able to define the true 'c' curve with confidence is an immense, and as yet, uncompleted task.

A mode of failure not covered by the Author was structural instability (buckling) and the influence of initial deformation on this particular capability.

With regard to the concluding paragraph, I read this as a cry from the heart, from an outport colleague to his headquarter's colleagues which could be epitomised in the cliché 'Give us the tools and we will finish the job'. I believe that so far as structural analysis and fracture mechanics techniques are involved, we do now have the correct tools. However, whether these costly tools are used to the extent idealised by the Author will depend upon how much the client is willing to pay for the advantages he may see arising from such an approach.

With regard to the resources currently devoted by the Society to the development of quality control procedures I tend to share the Author's opinions that these are not in proportion to the actual volume of staff engaged in practising quality control.

Finally, I would like to thank the Author for yet another interesting, provocative and entirely relevant paper which follows naturally from his previous paper on hull damages. Both these papers have appealed for greater need of exploitation of the Society's greatest asset, service experience. A request which we ignore at our peril.

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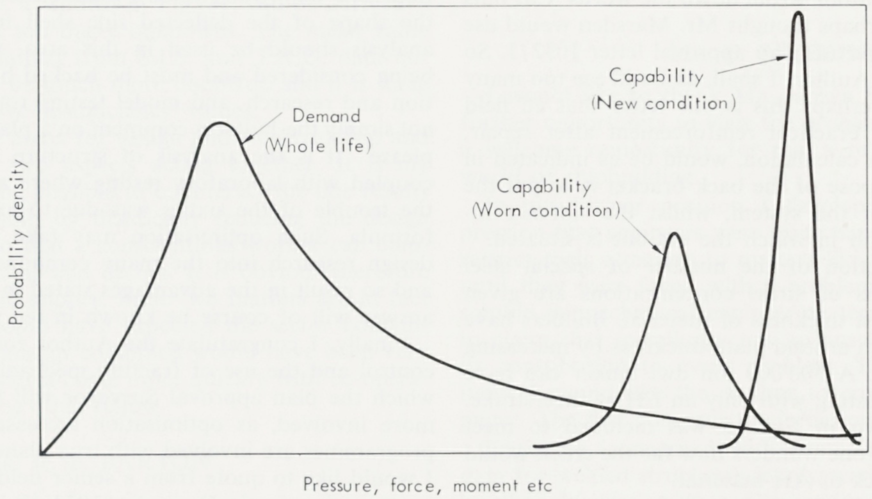


Fig. D 1

Probability Density Curves

Mr. W. Marsden

The Author has indicated in the paper the scope involved in current plan approval work. I know in Headquarters it is essential to have a good working communications system with metallurgists, quality control engineers and structural engineers. However, it was surprising to hear at a meeting last month, that in some areas, plan approval work is considered as being conducted from a totally separate office from field work. In fact some of the plan offices do not see Ship Letters or Noteworthy Defects Books and it appears field Surveyors do not see Plan Approval Letters. With optimisation and specialist designs, opportunity should be given for all to see written instructions, even if time is not always on our side.

I was pleased to read the Author's comments in Section 1 that 'survey procedures remain much the same as for smaller vessels', as this enables me to emphasize provision for staging systems in the large ships, to enable planned maintenance and minor repairs to be carried out at sea. Provisions for staging will enable the concept of monitoring surveys to be introduced, in which the physical capability of the Surveyor would not be the limiting factor. Further, the standard feedback from the monitoring survey would provide the detailed information which is required for future optimisation.

Referring now to the increase in the transverse stress mentioned by the Author in Section 2, I know he has done tremendous work in the investigation of the longitudinal bulkhead plating stress. It is not generally appreciated that some strakes of plating may be only 13·5 mm thick on a 300 000 ton dwt tanker and which may be subjected to the flange stress of the transverse frame of the wing tank, the axial loads from centre and wing tank, hydrostatic loads, shear loads and horizontal and vertical bending stresses. Longitudinal bulkhead plating has also a greater expanse than the bottom or deck plating in either centre or wing tank. It is to such structure that additional attention is required in quality control arrangements.

I heartily endorse the Author's comments that optimisation on fewer primary members coupled with a depth of primary member restricted to one plate width, results in larger deflections which can influence stress patterns in secondary members. A check on deflection should form part of all plan approval work, where optimisation is used.

I am sure when the Author wrote down the words 'Cut-outs for longitudinals', he perhaps thought Mr. Marsden would use that as an excuse to advertise plan approval letter 1032/1. So as not to disappoint the Author, I shall, as I still see too many of these defect cases. Perhaps this is one letter that all field Surveyors should have. Practical reinforcement after repair, if one is not involved in calculation, would be as indicated in Fig. D 2. The main purpose of the back bracket is to put the cut-out in the centre of the system, whilst the stiffener increases the panel strength in which the cut-out is situated.

Concerning the reduction of the number of special steel strakes, I hope positions of stress concentrations are given more preference than just thickness of material. Builders have and will be able to design around plate thickness by increasing secondary stiffeners, etc. A 500 000 ton dwt tanker can have AH material for deck plating with only an EH at sheerstrake. The DH material shown in Fig. 15 was included to meet earlier requirements and one wonders how far the crack would have propagated in a deck of AH material.

I hope the Author will excuse me if I say that readers should appreciate that the shaded area of Fig. 30 is only one of the areas where very high stress concentrations is likely to exist. It will be noted in the figure that the junctions of the crosstie are also areas where brackets are subject to distortion. The load transmitted by the crosstie is clearly shown by the shape of the deflected side shell transverse. Sub-routine analysis should be used in this area, where optimisation is being considered and must be backed by basic ship construction and research, and model testing routines. Optimisation is not simply the builders comment on a plan 'minimum scantlings please'. It is the analysis of structure by direct calculations coupled with laboratory testing where applicable. Remember, the trouble of the sixties was due to extrapolation of simple formula. Such optimisation may take more than a year of design research into the many components of a large tanker and so result in the advantages stated in Section 5.2. The final answer will of course be known in ten years' time.

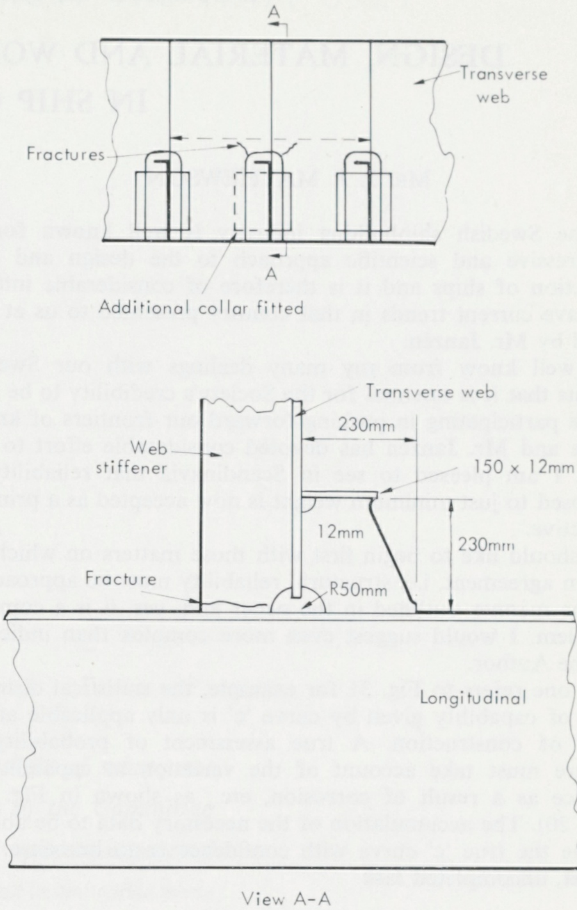


Fig. D 2

Reinforcement of Transverse web in way of cut-outs

Finally, I congratulate the Author for his theme of quality control and the use of fracture mechanics. These are areas in which the plan approval Surveyor will find himself becoming more involved, as optimisation increases. The finite element programmes are involved with true plane structure. Therefore, I would like to quote from a senior field representative's comments at a paper in December 1974, 'It is almost impossible in shipbuilding to effect large areas of plating without having some initial deflection due to the building process. It would therefore be helpful if all structural analyses were carried out

assuming that initial deflection exists'. I would like to ask Mr. Janzén concerning his conclusion, whether he has experienced the possibility of having the permissible tolerances of a quality control system incorporated in a finite element sub-structure routine. I know this has occurred in the case of simple plate buckling but other experience, perhaps as a result of Swedish ship research, would be welcome.

I have personally had many exchanges with the Author on day to day work and found him a most helpful and encouraging colleague. I know as far as exchange of information is concerned, the deficit is with me and so I hope my contribution evens the score a little. Congratulations on a very topical paper.

MR. A. K. BUCKLE

Several slides have been shown indicating how the addition of small brackets, etc., can reduce local deflections in way of stress raisers. This is in line with the general philosophy of minimising stress raising features but seems to be contradicted by lines 10 and 11 of paragraph 4.2 of the paper where good structural details are stated to be those which allow yielding. Perhaps the Author would care to clarify this point.

Reference has been made to the impracticality of Surveyors seeing all materials, fit-ups and welding in yards with a massive throughput of steel. In practice surveying must be based on some sort of sampling procedure in such yards. Is such sampling done on a systemmatic statistical basis?

About seven years ago the Society introduced a new section into the Rules—Section D 33 'Structural Details'. At the time it was anticipated that this section would incorporate the sort of advice that appeared in the 1967 edition of 'Detail Design in Ships'. Unfortunately the publication of such information has become something of a cinderella in practice. Does the Author favour such material being included in the Rules, or in a separate publication or kept semi-confidential for use only as consultancy background during plan approval?

It has been the practice in Rule Development to use varying probabilities of failure for different ship components, e.g. 0·5 per cent per ship year for anchor losses and 0·01 per cent per ship year for complete fracture of a solepiece. Bearing this in mind, and remembering the amount of structural redundancy built into a ship, it is not clear how real values for reliability of the ship as a whole, as shown in Fig. 31, can be derived—there seems to be just too many unknowns. The mean value can, of course, be obtained from S.I.S. and T.R.O. data but the deviations seem to be much more doubtful, and it is these that are needed to derive 'insufficiency' values.

Fig. 4 has two large areas for design and workmanship and a small area for the two aspects acting together. It is my experience that it can be possible to counteract the due results of dreadful workmanship by drastically reducing the design nominal stress and also that a conscientious worker can compensate for quite a lot of bad design. It is obvious therefore that, as the paper's title implies, these two matters are in fact interdependent so I would have expected the sector containing design and workmanship combined would have been very much larger. Improvements come more quickly with co-operation than by blaming the other fellow.

MR. R. F. BARTON

Many thanks to the Author for once again drawing attention to metal fatigue as a consequence of bad or indifferent detail design and as an important factor to be considered in the general problem of ship structural design.

As the Author has implied, modern analytical techniques in the structural field are extremely comprehensive and can, according to the degree of model sophistication, predict not only general stress levels but also stress concentrations. The very rapid advancement in the methods of calculation of applied stresses, however, does not appear to be matched with a corresponding appreciation of the importance of estimating allowable stresses, albeit the relative difficulty in determining load spectrums. This situation can, in certain cases, lead to uncertainty in the interpretation of computer output with a consequent reduction in the real effectiveness of the analysis.

I would, therefore, like to amplify several general points which are important in the field of 'first principle' stressing where field experience or Rule guidance is minimal or completely absent. One important factor in setting up a structural model is that of deciding upon the degree of fineness which gives the desired accuracy of load response. For example, where the intention is to use a standard S-N curve in a case of 'first principle' stressing care must be taken to ensure that the type of calculated applied stress is compatible with that used in the construction of the curve. Standard S-N curves are usually based upon nett area or nominal stress, the stress concentration aspect of the particular weld or notch shape being automatically allowed for. It should be noted, however, that standard S-N curves cannot make allowances for stress concentrations extraneous to those caused by the weld or notch geometry and in cases where the area under investigation is close to a cut-out, sudden change in section or a highly curved member the structural analysis should take such surrounding geometry into account.

With regard to corrosion fatigue there is still much experimental work to be done but results available suggest that it can be relatively important in the low stress, high frequency range where the so-called fatigue limit can be greatly reduced. Caution should be exercised when using normal 'Dry' fatigue data in this range for structural items exposed to corrosive elements. Stress corrosion due to exceptionally high residual stresses resulting from a particularly bad production sequence can also lead to premature failure. This again points to the importance of good detail design which is not only functional but which makes full allowance for the production aspect.

MR. F. WATKINSON

I would like to thank Mr. Janzén for providing me with a further opportunity to view the problems of ship construction; a welcome opportunity for me, a relative newcomer to this world of shipbuilding.

In my former position I developed the very strong impression that engineers were quite happy to have metallurgists restrict their attention to matters like material quality control until they were faced with a problem which was not readily soluble in mechanical engineering terms. I would be the last to claim that metallurgy has an answer in detail, concerning its contribution to every problem. There are, however, two items I would like the Author to consider in relation to the paper. The first concerns the initiation of fatigue failures.

Engineers, concerned with providing fatigue strength life data for welded structural details, were puzzled for some time by the observation that fillet welded higher strength steels showed no higher fatigue strength than that of similarly welded mild steel. After some prompting by metallurgists at the Welding Institute, the engineers, with some of their

customary mathematical wizardry, and, I suspect, a touch of engineering judgement, produced some equations, which resulted in the hypothesis that the low fatigue strength of fillet welded structures, common to mild and higher strength steels, would be consistent with the presence of sharp defects at the toes of the fillet welds. A size of the order of 0·4 mm was predicted as being critical. Needless to say, it was no problem for the metallurgists to find and identify defects which would meet this criterion. They were identified, in the case of welds made with manual electrodes, as small intrusions of refractory material originating from the electrode coating. They occur at the very edge of the weld bead and can be imagined as 'almost-melted' lumps of coating which become trapped under the edge of a freezing wave of weld metal as it solidifies on the shore of the parent steel. The intrusions were observed to be angular and ranged in size from 20 microns to about 0·5 mm, and the largest sizes were observed sufficiently frequently to conclude that they would be present even in relatively short weld lengths, say 50 mm.

Various means of removing these intrusions, or negating their effect, have been demonstrated. Most of the methods have resulted in substantial increases in the fatigue strength of fillet welded higher strength steels. Most of the methods involve some change in profile at the toe of the weld, as well as the removal of the intrusions, and thus there is some reduction in the overall geometrical stress concentration at the toe in addition to the removal of the metallurgical stress concentration.

When Mr. Janzén writes of TIG dressing, and grinding, as means of improving weld shape, it is necessary to realise all that is being achieved by these operations. TIG dressing melts and effectively removes the intrusions as well as improving shape, although the method, where it is practicable, is not absolutely reliable since, if handled incorrectly, it can introduce defects of its own. Grinding, on the other hand, can be completely successful provided it is specified so as to remove metal from the toe of the weld, to a depth of at least 1 mm below the as-welded toe. Profile improvement by grinding will not give the maximum improvement in fillet weld fatigue strength if it fails to undercut the toe by this minimum of 1 mm. While this 'undercutting' operation will clearly reduce the net section area of that part of the structure, it will be more than compensated by the increased fatigue strength.

The second point I wish to discuss is one where the Author has effectively asked for the metallurgist's assistance, and I can only assume that the 'Swedish experts' who were consulted were not aware of the relationships between fracture toughness and metallurgical microstructure which have been progressively accumulating in recent years. The Author, unfortunately, does not give enough information on page 7 of the paper to enable a positive answer to be given to the question, 'Why does HT80 steel show an increase in brittle fracture initiation temperature when the welding heat input is increased, whereas HT60 steel does not?' Well, the answer to the question no doubt can be expressed in terms of the formation of a particular type of microstructure at the slower cooling rates in the higher strength steel, resulting from the microalloying content of that steel. In the lower strength steel, because of its lower alloy content, the same type of microstructure is not formed at any cooling rate within the range covered by the data presented in Fig. 13. The fact that the lower strength steel has poorer toughness over the whole range of cooling rates is no doubt due to generally coarser ferrite and carbide constituents in the microstructure, compared with the higher strength steel.

Such microstructural differences are important, because of the differences in mechanical properties which result from them. It is necessary therefore that this type of information is interpreted in relation to steel composition and type, if the designer or engineer is to use the information sensibly and make the right decisions.

Mr. A. C. WORDSWORTH

I must congratulate the Author on a most interesting paper on a difficult subject. I was particularly interested in his remarks on the fatigue aspect as I have long felt that this has been given too little attention, particularly when one considers the large number of failures due to fatigue as demonstrated in Fig. 4. The Author's remarks about the desirability of doing actual fatigue tests are particularly opportune since a 50 ton fatigue testing machine has recently been installed at the Society's Research Laboratory at Crawley.

I would like to make a few comments on details of the paper which I hope the Author will not consider too niggling. Fig. 17 shows data taken from BS153. I regret that I have had no opportunity to check with this Standard but the headings at the top and bottom of this figure imply that 2×10^6 cycles is the fatigue limit of a welded joint whereas I had been under the impression that such joints had virtually no fatigue limit. Can you comment on this. In passing I would mention the absence of a stress figure in the table for a Class B weld and my dismay at seeing stresses given in kp/mm^2 in Fig. 17 alongside stresses in kg/mm^2 in Fig. 18.

In Section 4.4 the Author mentions the possibility of reducing a finite element mesh size until the weld itself is scrutinized. While this has been done in the past to try to predict crack propagation rates it was done using rather idealized weld profiles and not the irregular shapes shown in Figs. 18 and 27 which occur in practice and which are difficult to describe geometrically. I would suggest that, at least until all the crack propagation criteria are quantified, a more realistic approach might be to use either finite element or experimental stress analysis to determine the stress distribution which occurs at the boundary of areas of the structure at critical points. The size of these areas would be chosen to be small enough so that test specimens of them could then be made and tested at full scale with the fatigue testing facilities available. This might be done at the Crawley Laboratory.

Fig. 24 emphasizes the dangers of fatigue testing small scale specimens and directly transferring the results to a full scale structure although it may be possible to transfer small scale results provided the crack propagation criteria are taken into account. Fig. 24 is also interesting in that while the fatigue stress calculated for 9·5 mm plate agrees reasonably well with the stress given in BS153 the much lower stress for 30 mm plate does not. Possibly BS153 which applies primarily to steel girder bridges should be used with considerable discretion for this type of joint in the thicker plates. Incidentally, the plate thickness in Fig. 24 appears to have been misprinted as 3 mm rather than 30 mm.

Finally, I must also mention that the Research Laboratory has a technique for measuring the residual stress which the Author lists as constructional defects in Fig. 2. This technique might be used to try to quantify this particular defect.

WRITTEN CONTRIBUTIONS

Mr. F. N. BOYLAN

I must first of all convey the Association's thanks to Mr. Janzén for coming such a long distance in order to present his paper to us and I congratulate him on the content of the paper.

As he has stated in his title, the problem being reviewed is certainly complex and has up to the present been considered generally as a group of separate although related topics. There is no doubt that in recent years the accent has been very much on the development of techniques of analysis and only now are the problems of production beginning to receive some measure of the attention they deserve.

A very valid point is the tendency for most quality control systems employed in shipbuilding to be used as a means of achieving a general standard of quality, of making the workforce quality conscious—and aware of the attendant danger of repairing defects which would be much better left alone. Tolerance specifications and the related specifications of action to be taken when tolerances are exceeded, have obviously been drawn up within a very flexible framework. However, the Author suggests some fairly complex problems will have to be solved before more suitable standards are formulated.

The Author has succinctly reviewed the state of the art as it now stands in each of the individual components making up the total design and construction problem and perhaps we can look forward to further papers from members of the Association, outlining ideas on the form of a suitable comprehensive plan approval procedure which will integrate these components.

Dr. G. MOWATT

I would first like to add my congratulations to Mr. Janzén for producing a paper which obviously required a considerable amount of effort.

Mr. Janzén has discussed one of the more difficult problems facing the naval architect, that of the design of structural details. The problem is to define the load carrying capability of such details, and to determine the factors which influence the dispersion of the capability around a mean ideal value. I would like to direct my comments to the general philosophy behind structural reliability analysis by which the probability of failure of structure can be predicted.

The concept behind reliability or its converse, the probability of failure, is to introduce a quantitative measure in terms of which the safety and reliability of various parts of a structural system can be defined, compared and a uniform safety of the system assured. In general, the method of assessing the strength of ships has been based on a deterministic maximum load and minimum strength approach. In Fig. 31 Mr Janzén has shown the opposite extreme approach, that of a strict probabilistic concept where the demand and capability are represented by statistical distributions. The overlap between the tails represents the possibility of failure shown by the area to the left of zero on curve C-D in Fig. 31.

With regard to the capability distribution, the condition may be defined at two levels; that of damage and collapse. If we use elastic theory in our structural model, then the overlap will represent the possibility of damage, whilst if we use plastic and large deformation theory, then one can determine the possibility of collapse.

In Figs. 1 and 2 of the paper, Mr. Janzén has shown the process of construction and the possible defects in material qualities which influence the dispersion of the capability curve as shown in Fig. 31. Unfortunately, there is a dearth of information of the distributions associated with these individual components. In addition the mathematical models necessary to consider all the possible variations in the strength analysis is complex and hence time consuming in evaluation. The nett result is a difficulty in establishing sufficient information to adequately represent the capability distributions. Whilst sufficient information may be available to determine the general shape of the distribution, the fit to the 'tails' of the curves may be poor, and unfortunately, the tails of the distribution are critical if a full probabilistic approach is to be adopted.

An alternative 'middle road' between the fully probabilistic and existing deterministic approach exists. This middle road, whilst deterministic in nature, takes account of the statistical nature of the problem by considering the load and strength values to be characterised by the mean and standard deviation (s.d) parameters of the distributions. Whilst such parameters may still not accurately define the tails, they will give a general representation of the bulk of the distribution. Knowing the standard deviation, a parameter 'α' can be set such that a band '$\alpha \times$s.d' from the mean will contain say 95 per cent of the distribution. A load factor 'β' can then be established as shown in Fig. D 3 which relates demand to capability. Whilst such an approach is intuitively the same as our present deterministic method, it does consider the statistical nature of the distributions. In addition, since the approach calls for improved information on the criteria which affect capability, one can introduce quality assurance tolerances for both material standards and construction imperfections based on the effect of the variations on the spread of the capability distribution.

As our knowledge regarding the variations in material and constructional parameters improves, then the next step is indeed to go to a full statistical approach as suggested by Mr. Janzén. However would Mr. Janzén like to comment on this simplified approach suggested above as a suitable intermediate step towards a full reliability analysis of ship structures.

On a minor editorial note in Fig. 22 of the paper the same symbol is used for material without defects and material with porosity. Could the Author please clarify this point.

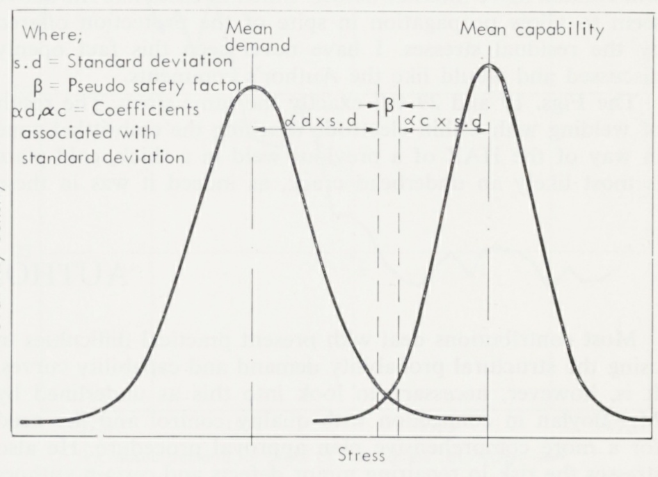


Fig. D 3

Probability Density Curves

Mr. O. NILSSON

Being the outport representative to LRTA in Sweden I have particular reason to thank Mr. Janzén for his willingness to translate and present this paper. As a member of a ISSC committee working on the significance of various imperfections on hull structures he is familiar with not only stress analysis and design aspects but also constructional problems and is therefore well fitted for giving a lecture of this kind.

The Kanezawa lectures as well as G. Johansson's work for his doctor's degree contain much useful information about ship failures and current laboratory work for establishing the reasons. It is a pleasure to read the Author's comments on this literature. It is understandable that he concentrates on problems connected with ships' behaviour in service and is less interested in constructional problems and that therefore hydrogen cracking, hot cracking and lamellar tearing are covered more in passing or not at all.

The following detail comments are offered: Charpy test results can be improved by alloying the weld metal, but the effect of such deposited metal may well be increased hardening. Electrodes suitable for high tensile steels may not always be the best for ordinary grade A quality. The low curve for the HAZ as shown in Fig. 12 is therefore not the only alarming factor to consider.

Professor Nibbering has studied the effect of the variation in stress gradients as shown in Fig. 14. He is a believer in practical wide plate tests. Whilst it may be true, as often heard, that a brittle fracture has never started from an automatic weld I have come across a few cases, where a fracture has gone *into* a weld and continued to propagate there. Mr. Janzén in fact shows such an example in Fig. 15 without commenting upon it. When the fracture coming up the ship's side of A steel meets the EH steel in the sheerstrake it is arrested but, as seen in the sketch, it has still power enough to overcome the residual stresses around the vertical butt and continue up to the deck through the weld. Joining butts welded on board between sections in this position are normally manually laid in the vertical upward direction. Charpy tests for this type of weld in laboratories show very bad results and are certainly no better when laid in practice, not even for basic coated electrodes. The similarity of an automatic high heat input weld is striking, the difference is that the manual weld will contain more internal defects which in cases like the above seem to allow propagation in spite of the protection offered by the residual stresses. I have never seen this fact openly discussed and would like the Author's comments.

The Figs. 19 and 29 tell exactly the same story. The result of welding with a thin electrode touching the embrittled zone in way of the HAZ of a previous weld in a thick, cold plate is most likely an underbead crack, as indeed it was in these two cases. Repairs with thin electrodes are very common at the Owner's request and I agree with the Author that such repairs should be avoided. I will emphasize, however, that pipelines are being welded in Siberia as well as in Alaska under conditions much worse than in a shipyard and that provided therefore proper precautions are taken, such welding may take place.

According to the Welding Institute a gap in a fillet weld must be very narrow in order to avoid hydrogen cracks from the root. The fatigue tests quoted by the Author show that the best results are obtained with a wide gap. Could the Author give some words of consolation to a confused Surveyor?

It is agreed that the modern statistical approach may be a useful tool for keeping records of structural efficiency, but care must be taken when applying this. Road traffic statistics do not prevent the police from doing their jobs in the case of, say, black ice on the road surface! In the same way statistics should not hinder the outside Surveyor from doing his duty in the shipyards, when necessary.

The most interesting part of the paper is in my opinion the conclusion, in which the Author summarises and looks ahead. He refers to the two level approach, one a workmanship standard, the other a fitness for purpose standard as presented in the current British Standard proposals for pressure vessels and the Swedish suggestion of dividing a ship into different quality zones. To suggest a combination of the two, as the Author does and as Mr. Harrison of the Welding Institute does also is perfectly in order, provided it is realised at first that the areas intended for this approach must be calculable. When, therefore, the Author proposes a solution in which work at present covered by the outdoor Surveyors should be taken over by the plan approval Surveyors I cannot follow his reasoning. There are a number of factors not known by the plan approval office which must be taken into consideration and as long as all standards in this field are arbitrary they must be dealt with by the Surveyor on site together with the production and control departments of the shipyards.

Responsibilities of the outdoor Surveyor are clearly laid down in the Rules and the Instructions. For example the questions on the first page of the First Entry Report are to be answered by the Surveyor responsible for the ship. If it is the Author's intention to interfere with building processes which are unknown to him and his colleagues then he is not only travelling too fast, he is on the wrong trail.

It is tempting to take up further discussion on the many more points raised by the Author, but I am afraid I have already broken the rules for contributors by submitting a lengthy contribution and must finish by again thanking the Author for his very fine paper.

AUTHOR'S REPLY

Most contributions deal with present practical difficulties in using the structural probability demand and capability curves. It is, however, necessary to look into this as underlined by Mr. Boylan in connection with quality control and the need for a more comprehensive plan approval procedure. He also stresses the risk in repairing minor defects and certain authors go as far as to consider about 50 per cent of the weld repairs carried out today are unnecessary.

Dr. Mowatt underlined the difficulties of the probability approach by mentioning the tails of the capability distributions and pointed to the practical middle road approach which, no doubt, will frequently be used in the future. Everyone would be satisfied with a curve covering 95 per cent of the distribution as the theoretical inaccuracy in this approach is negligible.

The top point in Fig. 22 refers to material without defects and this should have been indicated differently from material with porosity.

Mr Nilsson of the Gothenburg outdoor surveying staff has raised several practical points in connection with the actual building of the ships and intimates that the Author is only interested in the final product, i.e. how the ship behaves in service, which is to a large extent correct.

However, as Mr. Nilsson in his daily work deals with welding problems, the Author has little to add to his detailed points and would agree that crack propagation within the weld itself is rare and not a significant problem. The Author also considers that problems associated with fatigue should be given priority and this would justify the wide gaps suggested.

It would appear that Mr. Nilsson does not entirely agree with the remarks regarding extended plan approval, i.e. a plan approval that also tries to take due account of possible constructional and erection problems. It should be pointed out that it was never the Author's intention to interfere with the building process or in the outdoor Surveyors' work. However, the Author feels that it is the duty of a Surveyor engaged in plan approval to emphasise the relative importance of workmanship and fit-up in particular areas, having initially considered the possible stress situation in those areas when the ship is in service. It is much easier for a plan approval Surveyor to see the stress picture as he has often performed a rather extensive structural analysis. Such guidance is more important today as we are working with more highly stressed designs, i.e. it is no longer the traditional deck and bottom areas that are highly stressed. It is not easy, therefore, to estimate the need and relative importance of high class work for the different parts of the hull without a knowledge of the total stress pattern.

The Author agrees that many factors must be considered and that the quality standards now used are only arbitrary, while the effective standard is that decided upon by the outdoor Surveyors on site in each individual case. However, the Author believes that we would all gain from a more rational approach and that a more detailed knowledge is needed of the statistical distribution of the capability curve, both for the initial ship and also for the corroded condition.

Mr. Mathewson said that this need will be more obvious with the development of a good quality control procedure, but considers that the accumulation of such data is an immense task. However, it will be necessary and the Author is pleased to see that the outports' need for extended knowledge and tools is realized.

With regard to the influence of the initial deformation on the buckling strength, it is understood that further investigations sponsored by the Swedish Government will be carried out by the Swedish Yards. There is at present a great need for more precise figures regarding particularly the influence of plate deformations on the final buckling strength.

We all need, and deeply appreciate, an exchange of technical knowledge, a point which Mr. Marsden touched on. In particular, the Author appreciates the information regarding problems associated with large ships and many of the points raised in Mr. Marsden's contribution refer to such ships.

The knowledge gained during the design and analysis of these ships is used in a somewhat simplified form in the design of the smaller ship of today. The limited costs and numbers of these small ships do not permit extensive calculations and the Author does not believe that the finite element calculations incorporating initial deformations will be carried out at the outports. However, it is undertood that such work has been done at headquarters and, as mentioned in connection with Mr. Mathewson's remarks, some work may also be carried out by the Swedish ship research organizations.

The trend against fewer primary members has also been noted in the design of new, smaller ships and the large ship analysis technique has also been useful here.

During the present, rather gloomy shipbuilding situation, the possession of a good calculation technique analysis would be a great asset, such as is illustrated in Fig. D 4. This is particularly applicable to owners of ships with no immediate prospects of a charter.

The Author was slightly surprised by Mr. Marsden's initial comments regarding communications between the plan approval staff and the field force. It is difficult to imagine that communication could be avoided between Surveyors with similar interests in ship design and building problems.

It is virtually impossible to avoid yielding of certain areas of a finished detail, but with reasonable design and production care, this will not lead to failure and it is assumed that Mr. Buckle concurs in this. The Author agrees with the remarks made by Mr. Buckle concerning a desirable sampling procedure performed on a statistical basis but it is doubtful whether this occurs in practice. Regarding 'detail design', the Author is not in favour of such material being included in the Rule Book as the need for detail design levels varies from ship to ship and in different areas. The work involved in producing a reasonable 'detail design book' and keeping this up to date would be tremendous. Much useful information can already be obtained by carefully reading the Noteworthy Defects Book and, if necessary, supplementing this with additional studies. There are certainly a lot of 'unknowns' before real values for the reliability of a ship as a whole can be ascertained, but these simply make the problem an even greater challenge

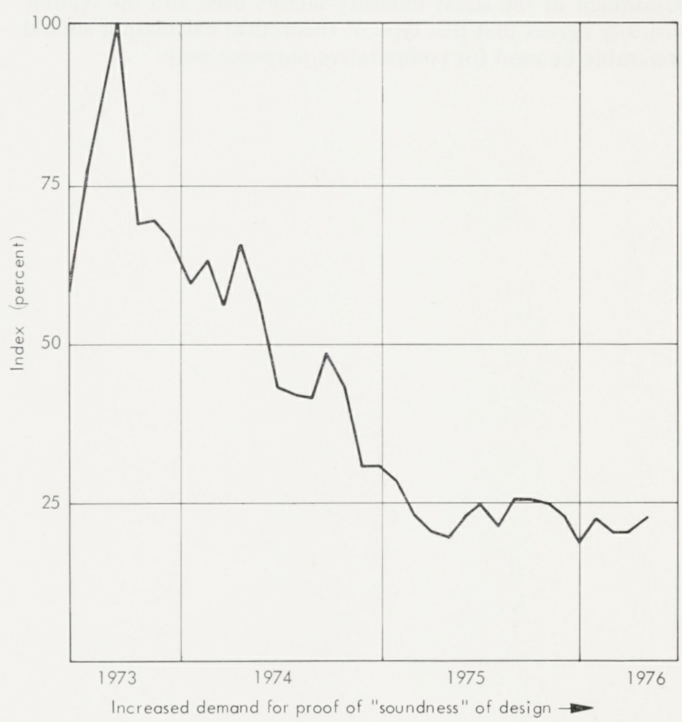


FIG. D 4

Freight Index (Tanker and Dry Cargo)

which we must accept and do something about. With the good co-operation that exists within the Society, this will be possible, but even with the best co-operation in the world, a conscientious worker cannot compensate for some of the poor details that have been approved in the plan approval office. The worker has no practical possibility today of significantly altering an approved detail design.

Mr. Barton gives some interesting and valuable comments regarding structural design and fatigue. The Author particularly noted the valid remarks regarding the use of standard S-N curves and associated nominal stress. Sometimes the degree of fineness of the structural model is carried too far. Mr. Barton correctly points to the basis for these curves, i.e. that the stress concentration aspect of the particular weld or notch shape is automatically allowed for.

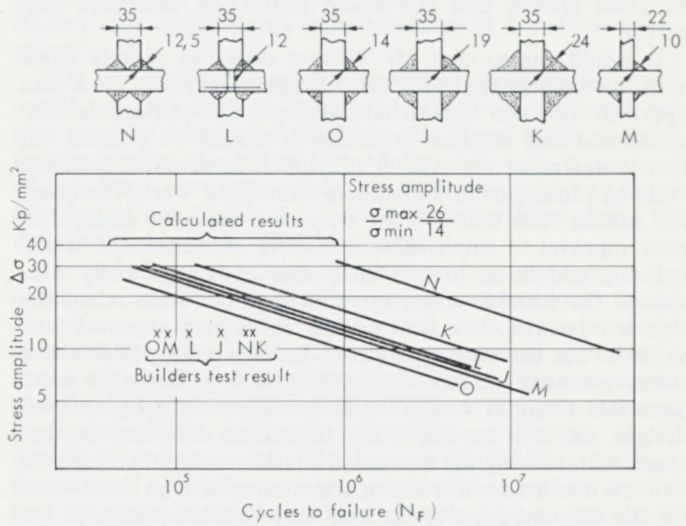
After a recent talk during the Scandinavian Welding Meeting in Sweden the Author was faced with similar comments to those of Mr. Watkinson. The Author therefore had the opportunity to test these on an audience of welding specialists and, judging from the response, it must be concluded that they are fully relevant and correct. The Author therefore hopes that the comments regarding TIG dressing and grinding are carefully read, as it is considered that the possible risks involved in the incorrect application of these techniques are not always appreciated.

Mr. Wordsworth discusses some of the shortcomings of our present theoretical calculation models for fatigue and crack propagation. These are illustrated in Fig. D 5 which shows the theoretical calculated values and corresponding results from full scale fatigue testing for various weld configurations. As can be seen, the values are not in full agreement but the relative merits of each are evident.

Closer agreement has been obtained recently by suitable adjustment of the stress intensity factors used and the Author basically agrees that this type of theoretical calculation should preferably be used for comparative purposes only.

The Author also believes that there will be opportunities for the extensive use of the testing facilities of the Crawley Laboratory.

Attention is drawn to some inconsistency in the nomenclature and some misprints which are regretted. The comment that a welded joint has virtually no fatigue limit is not fully understood but the comment regarding the need for considerable discretion when performing crack propagation calculations is concurred in.



$$\frac{da}{dN} = C(\Delta k)^m; \qquad \Delta k = f(a)\Delta\sigma\sqrt{\pi a}$$

FIG. D 5

Comparison of fatigue test results with calculated values for various weld configurations.

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IN-WATER SURVEYS

J. F. Wilson

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IN-WATER SURVEYS

by J. F. WILSON*

1. INTRODUCTION

This paper has been written to explain developments in in-water surveys. It is hoped the paper will be of assistance to the outport Surveyor who may be involved with both the assessment of a company applying to the Society for recognition to carry out in-water surveys and the in-water survey itself. A procedure has been suggested which is hoped will be of assistance to the Surveyor.

During the late 60s and early 70s it was thought that the growth in numbers of VLCC's would be so rapid that it would outstrip the availability of drydocks for repairs. It was also observed that even if there were an adequate number of drydocks there would likely be insufficient numbers conveniently situated on or near trade routes. The purposes in docking a ship are, basically, cleaning, painting and surveying. Whilst industry was developing means for cleaning and painting afloat, a working party was being set up at Headquarters to investigate the feasibility of surveying the hull of a VLCC lying afloat. In a drydock the Surveyor is able to see and if necessary touch any part of the bottom of the hull within the time of the docking survey which might take him say 45 minutes. Given the means of examining the immersed hull, the problems of doing the same job in-water were the inability of being able to touch the hull and the limited visibility which meant the job would take much longer than 45 minutes. If an attempt was to be made to examine every square metre of the hull in great detail, which is technically possible, it would prove prohibitively expensive for shipowners and the effort in setting up the system would be wasted. The approach adopted was to decide which items were most important to examine during a survey afloat and to allow industry to provide an economical means of carrying out this survey.

The docking survey reports of VLCCs classed with the Society were analysed in an attempt to identify the type and location of any damage occurring The pattern of results has suggested that the survey may be done on a selective basis thus cutting down the survey time. It must be remembered that this approach is a guide for the Surveyor, the actual survey being the responsibility of the Surveyor. If any damage is found then it will take longer to estimate its extent and a more thorough examination will be required.

Methods for carrying out the survey proposed by industry were evaluated including a report prepared by the Admiralty Underwater Weapons Establishment. Experiments were carried out to evaluate photographic and closed circuit television presentations and to evaluate the performance of various television cameras. Provisional Rules have been prepared by the Society which are explained in the paper together with an account of the trials witnessed by the Society and the suggested procedure for an in-water survey. As the survey afloat is only applicable if the hull can be satisfactorily maintained whilst afloat, i.e. cleaned and painted at the same time, current methods for carrying out this work are also included. To date, growth on the flat of bottom has not been a serious problem although isolated cases of extreme growth have occurred. If it should become a problem then there would appear to be no satisfactory means for removing growth economically and conventional drydocking would have to be used.

With the current trade recession causing lay up and cancellation of orders for VLCCs it is questionable at this stage whether shipowners will adopt in-water maintenance afloat. However, as operating costs are being reduced to a minimum there are strong indications that significant cost savings could be made over conventional drydocking. Relatively little experience has been gained at the time of writing this paper which, therefore, although primarily prepared for outport Surveyors, serves only as an introduction to the subject. Other methods may well be adopted if the demand for this service should increase.

2. SOME OF THE PROBLEMS

2.1 Visibility

Five metres below the surface may not seem far, but for man to survive he must be provided with his own life support system. If he is to work he should be able to see and be able to communicate with the surface support. He will also require specially designed tools and equipment. Life support communications and to some extent tools and equipment are available to a good standard but seeing is at best unreliable and appears to be the most difficult for man to control. If visual methods are adopted as a means for survey then it is as well to understand some of the problems involved with obtaining a visual image. It is not the Society's policy to have Surveyor divers, and it is considered that some form of a visual presentation and a report from a competent diver will be required for the Surveyor.

In-water visibility is limited to a few hundred feet under the most ideal conditions and is generally much less. During good conditions in the River Clyde area one may be able to see up to ten metres but a short while later only one metre. In the River Thames a diver usually cannot see his hand in front of his face mask.

Attenuation of light in water is caused by two independent physical processes, absorption and scattering.

2.1.1 Absorption

Water selectively absorbs light in a complex manner as a function of the wave length or colour of the light. Distilled water and clear oceanic water have the greatest transmittance in the blue-green region of the spectrum but this region reduces the light intensity about 4 per cent for every metre of travel. Other colours are more heavily absorbed and are almost completely eliminated after a few metres of travel. Contaminants in the water reduce light transmission further. They absorb more strongly in the short than in the long wave length region and therefore shift the peak of the water transmission curve towards the green or yellow region. This colour change is noticed in moving from clear oceanic water to coastal areas, i.e. a deep blue colour to a green yellow colour.

2.1.2 Scattering

Scattering of light in water, i.e. its deflection and hence change in direction of travel, adversely affects image contrast

* Ship Surveyor, Headquarters.

and is a great problem in in-water photography. If light is shone into fog it will be reflected by the fog particles and penetrate a certain distance. If the light intensity is increased, the ability to penetrate is not usually increased. Greater scattering takes place and one is often worse off than when one started. A similar situation may occur in water. To see further may not necessarily mean adding more light.

2.2 Size

Consider for example the problem of the in-water survey under the flat of bottom of the hull of a 325 000 tonnes dwt. tanker some 344 m in length and 53·3 m in beam (equivalent in area to about two football pitches). A complete television or photographic survey could cover a distance of five miles and monitoring the film take more than eight hours. If the speed of traverse of the television camera was increased the eye would see only a blur.

2.3 Damage

In order to design the system, basic parameters are required which must be provided by the classification society. Surveying depends on the judgement of the individual Surveyor and the decision to repair will be influenced by the severity and location of damage on a particular type of ship.

The main item likely to require repair is damaged plating, which may include a fracture and corrosion. The damage could be considered as requiring repair or being held over to the next drydocking. The problem was to establish parameters that would define the difference between the two. A guideline was therefore proposd suggesting that the minimum set-up in plating that should be detected be approximately 18 mm. Ships qualifying for in-water surveys should be fitted with an impressed current system from which it will be possible to establish if any serious corrosion had occurred by examining the log sheets.

2.4 Logistics

The time taken to complete the survey should be considered together with portability of equipment and ability to operate under varying conditions. It would be of advantage if the Surveyor was able to assess the condition of the hull at the time of survey without waiting for any results to be processed, i.e. photograph material.

3. TECHNIQUES

Means proposed by industry for obtaining survey information have included photogrammetric, television and acoustic methods. One or a combintion of these methods should therefore provide the information that enables the Surveyor to decide whether or not, as a result of the survey, the ship is in a satisfactory condition to continue trading for a further two or two and a half years and the notation IWS be recommended for inclusion in the Supplement to the Register Book.

3.1 Acoustic

Some laboratory tests have shown that when an acoustic transmitter/receiver was passed over a butt-welded steel plate with a 4 mm displacement at the butt, this displacement was clearly visible on a print out of the acoustic signal. The transmitter plate distance was 5 m. The proposed method consisted of constructing a boom, of length greater than the breadth of the largest VLCC, fitted with transmitter/receivers spaced at equal intervals apart. This boom would either be fixed to the seabed or be passed under a moored vessel. Any heel or trim would automatically be accounted for in the system. In addition there would be diver held closed-circuit television (CCTV) work and the measurement of shaft and pintle clearances. However, it is considered that the cost of such a system would be so great as to render it uneconomic.

3.2 Photographic

A system of in-water 'aerial' photography has been proposed whereby a series of colour stereo-photographs of the bottom of the ship can be obtained and analysed to measure any possible hull deformation. The cameras would be mounted on a boom and take a series of photographs as the ship passes over. Disadvantages of this method are that the vessel must be at a particular level of trim and the time required to develop and analyse the photographs is rather long. With this system there still remains the inspection of hull sides, etc., and clearance measurements.

3.3 Closed circuit television (CCTV)

The advantages of this type of system are:—

(i) real time is used enabling the surveyor to make his decision before the ship sails
(ii) information may be quickly and easily recorded.
(iii) commercial equipment is available and designed for in-water use
(iv) in conditions of poor visibility it has been found that the camera can see more than the diver
(v) the adoption of CCTV would probably involve the least capital expenditure and development costs for a contractor.

The disadvantages are:—

(i) the resolution is not as good as a photographic presentation
(ii) colour photography is not readily available
(iii) it is difficult to detect set-up when dealing with a two-dimensional effect
(iv) very good visibility is required to obtain a fair coverage.

3.4 Air

During trials it has been observed that exhaust gases from the diver's demand valve tended to collect in indents in the hull plating (where these existed) giving a clear indication of the presence of damage. This should be a very simple and inexpensive technique to apply. It has so far proved difficult to detect indents using visual methods.

3.5 Diver

One simple method proposed was to permit a diver, whose survey ability and experience was considered satisfactory, to carry out the survey and for the Surveyor to accept his report. This procedure was UNACCEPTABLE to the Society as the Surveyor was unable to verify for himself any of the findings and had to rely entirely on the report of the diver. The Surveyor would not be in the position of being able to give an impartial decision.

3.6 Summary

So that a meaningful inspection may be carried out a combination of CCTV, still colour photography, air release and diver's report with good two-way communication with the Surveyor, is considered as one method acceptable to the Society. The Surveyor on site must be satisfied with the visual presentation at the time of the survey.

4. IN-WATER SURVEY TRIALS

The Society has been involved with important trials in the harbour areas of Lisbon, Las Palmas, Greenock and Marseilles and the following account may be of interest.

4.1 CCTV trials

The principle of using a controlled vehicle, in association with closed-circuit television, was established at the Lisbon trials. The television was satisfactory in producing good pictures of gratings, etc., but due to tide and murky water the conditions were not good enough to permit a satisfactory survey of the sides or flat of bottom. This exercise also proved that a means of identifying areas of the immersed hull was essential and that surveys would have to be carried out at locations offering better in-water conditions than Lisbon. Accordingly, the outer bottom of No. 2 cargo oil tank of a 215 000 tonne dwt tanker was painted with white lines in dry dock to a system agreed with the Society and six months later examined in-water at Las Palmas.

The object of the Las Palmas trial was to examine the condition of the white lines and to evaluate three closed circuit television cameras including a new low light level camera. The tanker was at a ballast draught of 11·5 m when moored just inside the outer mole. The conditions for survey were excellent, a diver swimming on the surface with a face mask was able to examine from the water line to the turn of the bilge. When the vehicle was on the keel strake, the television camera was producing a good picture with only the available ambient light. One could see clearly 10–12 m on the monitor and it was observed that the white lines were almost complete. The time was approximately 1400h on a sunny day. It was found that the best time to carry out the flat of bottom survey was between 1000h and 1600h when one had a maximum benefit from sunlight. Unfortunately, without adequate mooring facilities this location has proved awkward for large ships to date, although the survey conditions are excellent and there are facilities under development for cleaning and painting afloat. The philosophy so far had been to survey at locations offering ideal conditions with the hope that other essential facilities would be developed in time. The next trial in Kames Bay, Rothesay, on a coasting tanker showed how air bubbles released from a diver migrated towards indents on the bottom of the hull and gave a clear indication of the presence of damage. It had been very difficult to detect set-up in the plating from photographs or television pictures when the paintwork had not been disturbed, and the release of air proved to be a simple and effective way of overcoming the problem. It was also observed that an air bubble was formed at the ridge of paint left by a docking block.

4.2 Complete survey

Another approach was to attempt an in-water survey at a repair quay where normal maintenance facilities were available and no delay would be caused to the ship. The conditions for visibility were likely to be poor and nothing like those experienced at Las Palmas. Marseilles was finally selected and an in-water survey carried out on a 70 000 ton dwt tanker inside the habour, at quay 121, the ship being at a mean draught of 4 m. She was cleaned afloat in the bay by divers using rotary brushes before she entered harbour for her 14-day refit period. The frame numbers were marked on the sides of the ship before the survey commenced and identification of the bottom was achieved with a system of rope grids as no identification lines had been painted on the bottom before the trial. The ropes were spaced 1·5 m apart with discs on the ropes indicating the frame number, port or starboard side, position of the keel and position between the frames (frame spacing 4·5 m).

A closed circuit television camera was mounted on a trolley operated by the diver and was detached from the trolley for surveying areas other than the bottom. Two-way communications were available between the diver and the surface support. A control cabin, which was moved along the quay as the survey progressed, contained the monitors and videotape recording equipment and also provided room for at least six people to view the result in comfort.

A complete survey was carried out on the in-water portion of the hull and recorded on videotape during the first five days of the refit period. Measurement of the weardown of the rudder pintle and stern tube was taken by a competent diver. Five sea inlets were blanked off and the sea valves cleaned successfully. Thickness measurements of an area of the bottom were obtained using an ultrasonics set adapted for use in water. The set remained on the surface with the probe attached to a long cable. The survey showed that the forward port anode was missing, a section of the port bilge keel was bent and missing, cavitation erosion had occurred on all the propeller blades and there was considerable paint detachment with insignificant corrosion in places. A slight set-up in No. 3 port wing ballast tank was reported by the diver but subsequent internal examination found no damage to the structure.

The ship was drydocked for 24 hours immediately after the repair period and the in-water survey findings were confirmed. It was also confirmed that there was no set-up in No. 3 port wing tank. As a result, the trial was finally agreed to have been successful for classification purposes and the ship could have been awarded the IWS notation. As already stated, the in-water survey was carried out over a period of five days. It should be noted, however, that the ship was unprepared, and had already been organised for a 14-day refit period. These considerations, combined with the experimental nature of the in-water part of the survey, meant that time was not critical. Had different circumstances been applicable, the in-water survey could have been completed in a much shorter time. A total of 12 hours of videotape of the survey was recorded which subsequently proved very difficult to replay in London. The machine used for recording was not available and the one hired in London, although similar, was unsatisfactory.

4.3 SCAN

One of the final trials was organised in the Clyde off Rothesay in November, 1974, to test a new survey vehicle SCAN. The trial was conducted on a coasting tanker (18 000 tonnes dwt) immediately after her drydocking. The vessel was built in 1955 and it was hoped that there would be areas of interest that would serve as a useful test for the CCTV cameras, such as pitting, indents and corrosion of welds. In actual fact the hull was in remarkably good condition and there were very few areas suitable for this exercise. The only possible items being some of the butt welds with slightly corroded areas. These were photographed in drydock but due to a fault in the SCAN vehicle it was not possible to obtain comparative in-water photographs.

The SCAN system mentioned above has been developed employing the techniques suggested in 3.6. CCTV is used to search for damage together with air released from divers. A

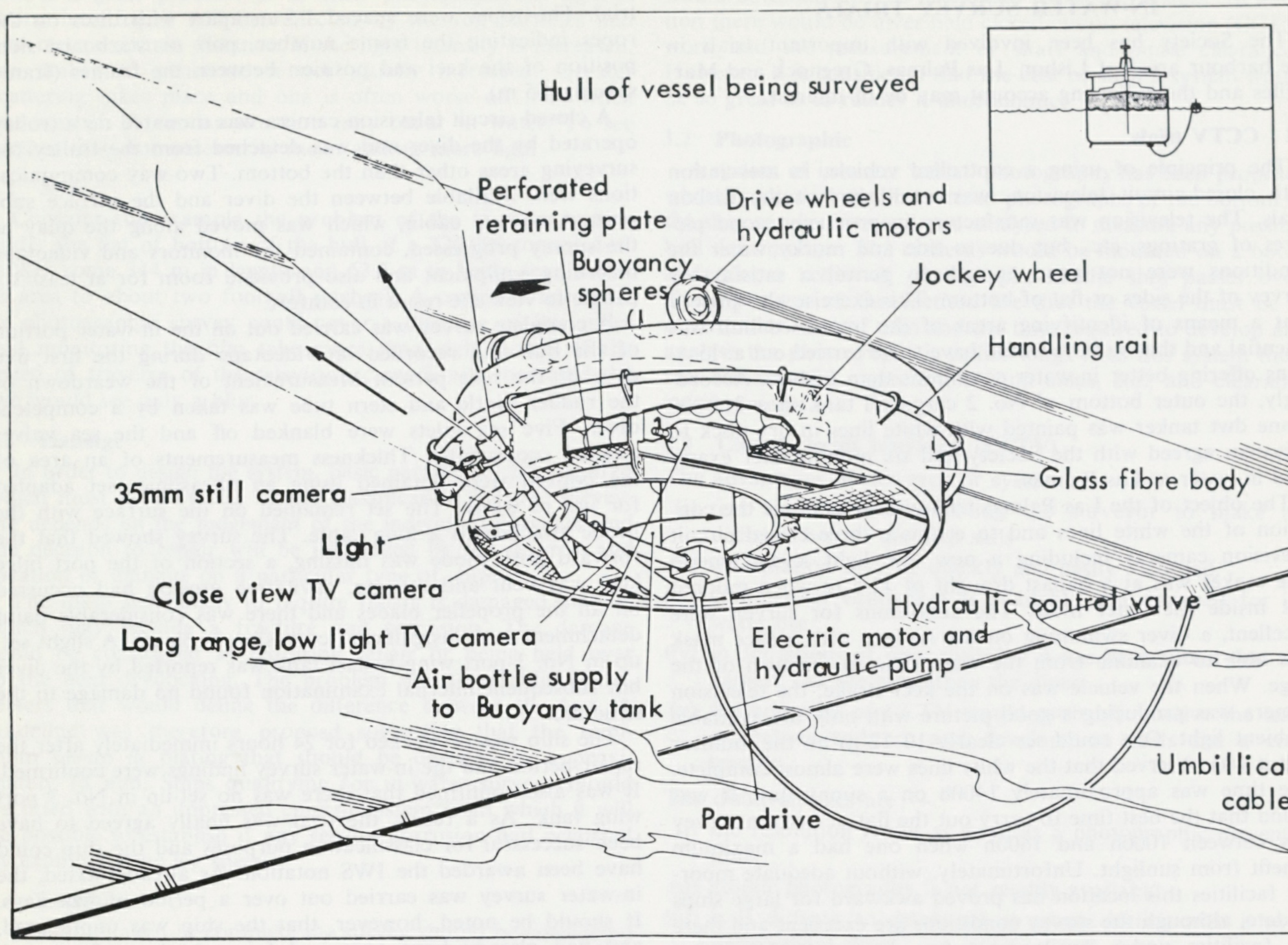

FIG. 1

In-water survey vehicle (SCAN).

diver may be sent to report and detailed colour information can be obtained from a still camera. The basic equipment of SCAN consists of a diesel generator, control console and submersible vehicle. Fig. 1 illustrates the vehicle which consists of a glass fibre reinforced plastic vessel providing a rigid platform on which is mounted a pan unit carrying two CCTV cameras and one photographic camera. It also houses a hydraulic system which powers two of the three wheels. The fibreglass structure accommodates buoyancy material to give the vehicle approximately neutral buoyancy when submerged and it forms a central chamber which can be filled with air to force the vehicle up against the flat of the bottom with a net upthrust in the region of 150 kg. The vehicle can travel up to 15 m/minute against a 0·5 knot current which has been found to be a maximum speed at which information can be assimilated from a TV screen. It will only operate on the flat of bottom of the ship (*see* Fig. 2). One CCTV camera is set at a shallow oblique angle which will display a trapezoidal area of about 10 m depth and 9 m maximum width in water of sufficient clarity. The second CCTV camera and photographic camera are set at almost right angles to the hull to show a smaller area in greater detail.

FIG. 2

SCAN in operation.

A standard vidicon camera with auxiliary lighting has been shown to have adequate sensitivity for giving acceptable pictures under a tanker hull. Measurements of reflectural light intensity under the hull has suggested that a more sensitive TV camera could be used without using auxiliary lighting. The near view camera therefore is a standard vidicon camera using a thalium iodide light. A low light level camera with a sensitivity of at least 50 times that of the vidicon camera has been used for the forward view camera. This eliminates the need to supply light for the forward view camera and the possibility of producing back scatter from light which gives a foggy picture. The CCTV cameras have remote control focus which is operated with the 35 mm still camera, from the surface control console. When the film has been used a second camera is fitted in place of the first camera.

4.4 Optical measurements

A quantitative assessment of the medium in which surveys were likely to be carried out had been a requirement from the start but there never seemed to be time during trials to do more than test the equipment being used. Various simple measurements had been taken but it was considered that some standard procedure was required which would produce basic data and which would also help assess any new sites for in-water surveys. The procedure should involve equipment readily available, cheap to transport and easy for the diver to use. It was therefore decided to carry out certain tests during the Rothesay trial and time was planned for this. The important information required was an assessment of the water quality and the measurement of light intensity around the submerged area of the hull.

4.4.1 Water quality

This should give some indication of the ability of light to travel in water. An elementary measurement is possible using a secchi or white disc. The disc should be adequately weighted and attached to a line calibrated in metres and half metres. The reading is subject to some variations arising from variable light, e.g. sun, overcast, angle of sun and any surface ripple or waves. The depth at which the disc disappears is noted together with the water surface conditions and incident light conditions either as sunny or overcast and by measurement with a light meter. A considerable improvement in secchi disc readings is gained by eliminating the water surface effect by looking at the disc through a diver's face mask. More than one person should take a reading as people's eyesight obviously varies.

The results of such readings are given in Table 1 below. A vertical reading in Las Palmas was in excess of 15 m, whilst a vertical reading in Marseilles was only 4 m.

Surface Water	Ambient Light	Surface Incident (Weston)	Disc Depth—Vertical	
			A	B
Choppy	Sunny no clouds	12·5	6 m	6 m

TABLE 1

Visibility measurements at site.

4.4.2 Light intensity

The object is to measure reflected light levels at intervals around the hull from the water surface to bilge and from the bilge to the keel. A light meter in a waterproof case is required, capable of giving readings at very low light levels. The light meter used was the Lunasix 3 mounted in a perspex waterproof housing the base of which was set at 45° to the

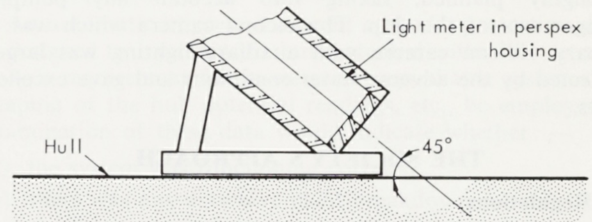

FIG. 3

Waterproof lightmeter.

plane of the meter. The base was positioned flat against the surface of the ship so that reflected light leaving at 45° to the surface of the hull was measured (Fig. 3). The surface incident light was measured at the same time as the diver

Water Surface	Ambient Light	Surface Incident (Weston)	Depth (metres)	Side of Ship	Flat of Bottom		
					Inboard	Outboard	Longitudinal
		12·5	0	Off	—	—	—
				meter	—	—	—
Choppy	Sunny	12·5	2	scale	—	—	—
	with no	12·5	4				
	clouds		Bilge				
		12·5	2		6·5	10·2	8·5
		12·5	4		5·2	9·0	6·3
		12·6	6		4·8	8·0	6·0
		12·6	8		4·0	7·1	5·0
		12·6	10		3·1	5·0	3·9
		12·6	5		5·2	8·1	6·5

TABLE 2

Optical measurements around the hull.

measured the underwater reflected light. Good communications therefore were necessary with the diver. The procedure adopted was to measure the reflected light off the side of the tanker at 2 m intervals from the surface to the bilge. This gave an indication of the reduction of light with depth and hence an idea of water clarity. Readings were taken on the flat of bottom from the bilge to the keel at 2 m intervals with the meter facing inboard, outboard and longitudinally for each reading. The diver must be careful to avoid shadowing the meter himself when taking the readings. The results obtained are given in Table 2 which were taken on the 18 000 tonnes dwt coasting tanker.

The readings obtained indicated that there was sufficient light around the hull for the low light level camera fitted to the SCAN vehicle to give satisfactory results. The pictures finally obtained were very poor and useless for inspection purposes. The divers were reporting a fall off in visibility and as the day was still very sunny the loss of visibility and poor performance of the camera was a mystery. It was subsequently discovered, however, that at about this time the ship was changing the dirty ballast water, obtained at the time of drydocking a week previously, and it was this dirty water which adversely affected the results. This was unknown to the shipowner's representative organising the trial survey and helps to illustrate that any future surveys must be thoroughly planned, taking into account any pumping arrangements of the ship. The second camera which was an ordinary vidicon camera with auxiliary lighting was largely unaffected by the adverse water conditions and gave excellent results.

5. THE SOCIETY'S APPROACH

5.1 Regulations

5.1.1 Requirements

The object and extent of the in-water survey for classification purposes is to provide, so far as practicable information on the condition of the ship and the machinery which is normally obtained from the drydocking survey requirements of Chapter C 1 of the Rules. For complete survey of the screwshaft, the requirements of C 12 are to be carried out. The survey therefore includes a visual examination of the rudder, propeller, hull, anodes, sea inlets, gratings and fastenings and the measurement of weardown in the rudder bearings and stern bush. Sea valves should be examined once every four years or five if the year of grace is included. In the past a certain percentage have usually been examined at every drydocking of the vessel. It is possible therefore to examine all the sea valves at the special survey docking but, if the owner requires, a certain number may be examined at the time of the in-water survey.

5.1.2 Application

In-water surveys apply to the following: —

(i) Initially to intermediate surveys only, not to special surveys. In practice this could mean an extension of the drydocking interval to four or five years with an in-water survey between dockings. A new survey record IWS (with date) has been introduced and will be entered in the supplement to the Register Book.

(ii) To ships less than 10 years old with a beam greater than 38 m. Passenger ships are excluded. (It is considered uneconomic to have in-water surveys on vessels with a beam less than 38 m.)

(iii) To ships which have a suitable high resistance paint applied to the underwater portion of the hull and an approved automatic system of impressed current external cathodic protection.

(iv) To ships complying with the provisional Rules 1–12 detailed below at the time of survey.

5.2 Provisional Rules

The following rules have been proposed for consideration during the design stage of new ships or for adaptation to suitable existing ships in order to permit complete surveys to be effected in water.

5.2.1 Stern tube

Means should be provided for ascertaining the clearance in the stern bush and, where fitted, the efficiency of the oil gland with the vessel afloat. Where it is required, the method for taking poker-gauge readings should be submitted for approval. Many VLCCs are already fitted with stern gear which permits weardown to be measured with the vessel afloat. With the ship at a light draught, exposing say half the propeller boss, it is a simple job to measure the weardown with the poker gauge. Some types of stern gear such as SIMPLEX allow the reading to be taken in water. The removal of the plug for the poker gauge does not normally contaminate the sealing arrangement, but the Surveyor should check this for the particular gear fitted.

5.2.2 Rudders

The closing plates of rudders should be designed to facilitate measurement of the bottom pintle and bush clearances and for verifying security of the pintles in their sockets. A tanker with a two pintle arrangement was fitted recently with portable closing plates in way of the pintles. The plates which were bolted to give a flush fitting, were large enough to allow the pintles to be fitted and removed when required. A lifting eye was made which could be screwed into the plate at the time of removal. Access was therefore available for a diver to measure the pintle clearances.

The pintle nut was enclosed by a brass cover plate filled with grease. In order to check at the periodical survey that the nut had not moved, a small removable plate was fitted in the skeg directly below the nut with a plug in the brass plate above the opening. This could be removed by a diver who would then insert a specially made 'dip stick' to measure the distance from the plate to the nut. The original measurement taken in drydock would be known so any variation would be noted thus indicating any fall in the nut. However, it might be possible for the nut to slacken off and the pintle to rise, which would not be indicated by the reading. Therefore a simple batten was made which could be placed across the top of the pintle and bearing by the diver. Any vertical movement in the pintle could then be measured from the batten.

5.2.3 Identification

Sea openings: The position and identity of sea inlets and discharges should be marked with a vertical line 3 m long above the openings for easy location by the diver.

Deck edge: Frames and bulkheads numbers should be marked at weather deck edges and the position of the oil-tight bulkheads marked near the load water-line to permit identification from a launch. Welded beads for identification are permitted for marking such items as the ship's name, draught

and load line provided certain precautions are taken during welding. It is, however, considered that welded beads which may promote cracks in way of weld defects should not be adopted for the considerable amount of general identification marking required.

Flat of bottom: Identification on the flat of bottom area should be provided at the time of survey. This may be achieved with a rope and grid, painted lines put on during a previous drydocking, acoustic or other methods. The rope grids are usually provided by the contractor and take considerable time to erect and move down the ship as the survey progresses. Trials have been carried out using an acoustic system which indicates that transponders could be placed at known positions on the bottom of the hull by divers. A hydrophone either held by the diver or fitted to a vehicle may be used to obtain the distance from each transponder and using two transponders it should be possible to navigate on the flat of bottom taking cross bearings.

Where it is proposed to paint lines on the bottom of the ship it is the Society's intention to develop and adopt a standard system of marking. Any proposals, therefore, should be agreed with the Society. The surveys carried out to date have shown that a limited amount of marking would be the most effective way of doing the survey. Fig. 4 shows the scheme for an oil tanker which involves: —

1. a broken line about 6 m inboard of the bilge keel,
2. a broken line at the keel,
3. the positions of oil-tight and wash bulkheads marked on these lines,
4. the tank numbers marked at the tank boundaries.

All lines should be marked alongside and follow welded seams and butts.

5.2.4 Plan

The shipbuilder should prepare an in-water survey plan. This plan should show all external details of the hull, together with an indication of the bulkheads and where applicable the agreed system of marking the bottom. A scale of 1:200 has been found suitable for use on the site by the Surveyor.

5.2.5 Photographs

Photographs of the following items, preferably in colour, should be provided either before the ship is launched or at a drydocking: —

1. bottom rudder pintle where clearance readings are taken,
2. propeller boss and rope guard showing plug for poker guage,
3. main sea inlets,
4. impressed current anodes,
5. athwartship thrust units, if fitted,
6. additional items as may be considered necessary.

Principal dimensions and location, together with a suitable scale are to be indicated on the photographs. Copies of the photographs are to be available on board ship and are also to be provided for the Society's Head Office records.

5.2.6 Draught

The vessel should have a draught of less than 10 m for an in-water survey with an ideal draught in the region of 6 m having little or no trim. If a special ballast condition is required then this should be submitted to Head Office for the approval of longitudinal strength. Usually, however, the trim and stability booklet contains a suitable harbour condition which may be used.

5.2.7 Impressed current log sheets

An automatic impressed current cathodic protection system is designed to maintain the hull potential within the set range. It is a requirement for in-water surveys that a system of log keeping of the hull potential readings, etc., be employed. An examination of these data would indicate whether: —

(a) the system was functioning correctly,
(b) there was any extensive paint breakdown,
(c) there was any significant corrosion.

Copies of the log sheets should be submitted to Head Office at least one month prior to the in-water survey. Each log sheet which covers one month is filled in daily and the notes below should help in understanding the readings on the sheets. It is useful to check the previous 18 months' readings.

1. The hull potential should be in the range +250 to +300 mV based on a zinc reference cell.
2. Values greater than 300 mV indicate under-protection.

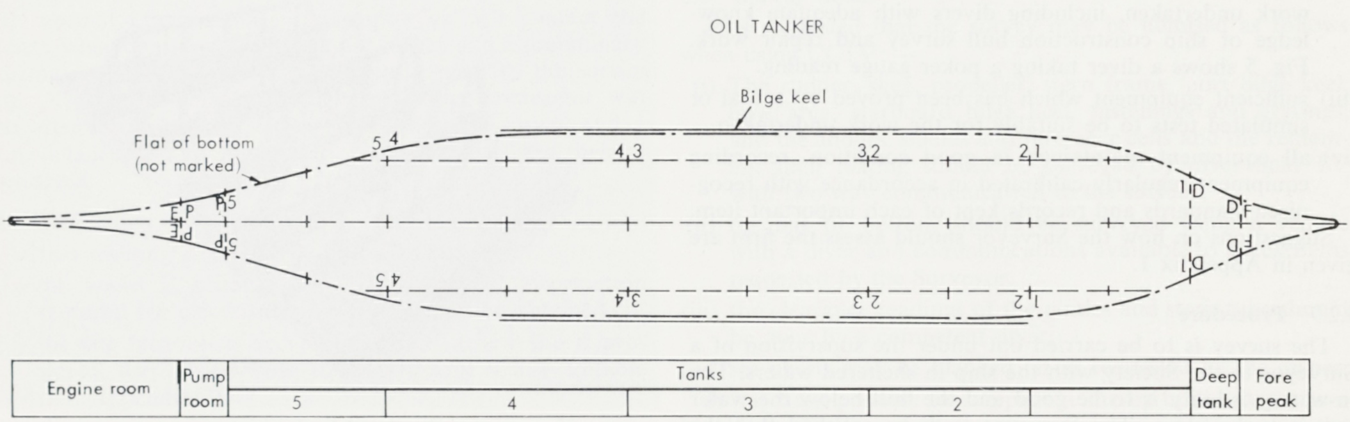

FIG. 4

Markings on flat of bottom.

3. The potential of a normal unprotected hull based on zinc reference electrodes is in the order of 500 to 600 mV and prolonged protection at such values may result in corrosion at breaks in the hull coatings.
4. Values less than 250 mV indicate over-protection.
5. Values less than 200 mV may result in damage to paint coatings. The resistance of paint systems to cathodic protection can be considered to increase in the following order: bituminous aluminium, chlorinated rubber vinyl, coal tar epoxy and epoxy.
6. The majority of cathodic systems are capable of providing current densities of the order of 30 mA/m^2.
7. If, for constant conditions of sea temperature, resistivity and vessel's speed, the current increases while maintaining the hull potential then it can be assumed that there is a significant loss of paint.
8. The two major causes of failure of the cathodic protection system are:—
   (i) the breakdown of the dielectric shield around the anode resulting in spurious readings,
   (ii) the loss of platinum from the platinum titanium and platinum niobium anodes. This is often caused by the scrubbing in water where divers have directed their machines across the shields and anodes.
9. Attention is drawn to plan letter PL 1014/1 for further explanation of the impressed current system.

It is recommended that the system is switched off when diving operations are being conducted under the ship as there is possible danger to divers approaching within 1 or 2 m of impressed current anodes which are in operation. Development is still required by industry for producing means of maintaining and servicing the protective shield of the anode and for changing anodes in water.

5.2.8 **Contractors**

The in-water survey is to be carried out by firms recognised by the Committee.

For the purpose of recognition the firm will be required to demonstrate to the satisfaction of the Surveyor that it has:—

(i) a clearly defined management structure with responsibility for each activity indicated,
(ii) sufficient staff suitably qualified and experienced for the work undertaken, including divers with adequate knowledge of ship construction hull survey and repair work. Fig. 5 shows a diver taking a poker gauge reading.
(iii) sufficient equipment which has been proved by actual or simulated tests to be suitable for the work undertaken,
(iv) all equipment maintained in good condition, recording equipment regularly calibrated in accordance with recognised standards and records kept of each important item.

Suggestions on how the Surveyor should assess the firm are given in Appendix 1.

5.2.9 **Procedure**

The survey is to be carried out under the supervision of a Surveyor to the Society with the ship in sheltered waters. The in-water visibility is to be good and the hull below the water line is to be clean. The Surveyor is to be satisfied that the method of pictorial presentation is satisfactory and that the information obtained enables a reliable assessment to be made of the condition of the hull. The services of a diver are also

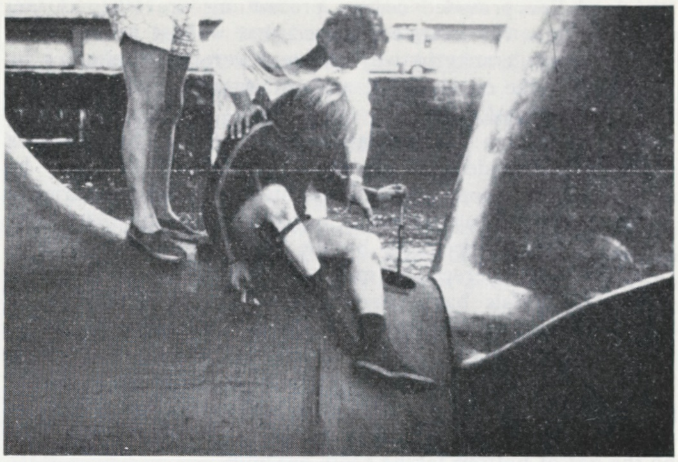

FIG. 5

Diver taking poker gauge readings.

to be available for this purpose and there is to be good two-way communication between the Surveyor and the diver.

5.2.10 **Non-compliance**

Proposals to carry out an in-water survey on ships not fully complying with the provisional Rules will be specially considered.

5.3 **Recommendations**

The following are recommendations and do not constitute rules:—

Gratings: To facilitate cleaning, gratings should be hinged so that they are easy for a diver to remove and replace.

Closing appliances: Blanks or other closing appliances to enable the sealing of sea connections and water boxes should be specially prepared and carried on board to permit valves and fittings to be stripped down, inspected and, if necessary, repaired from on board. Fig. 6 shows such a blank.

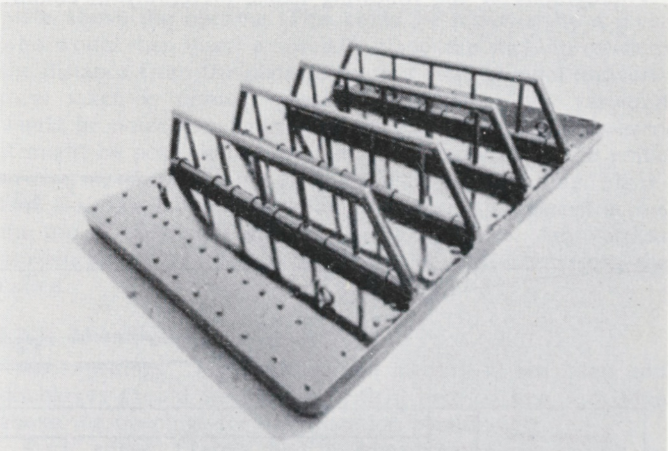

FIG. 6

Closing appliance (2·3 m×1·2 m)

It is understood that blanks are often used by ship operators. They should be strong enough to withstand external pressure as it is possible that the grating would not withstand the pressure by itself. Some shipowners are requesting approval of these blanks by the Society. Our involvement in such cases covers only the structural approval of the blank with a test to twice the rated working depth of one blank of each design. The Society is not involved with the operation of the blank. If a shipowner uses an approved blank and it is incorrectly fitted resulting in flooding of a compartment then this is the owner's responsibility and not the Society's. Where considered necessary, combined pressure release and drain valves or cocks, with permanently attached blanking arrangements should be fitted on the sea connections in accessible and visible positions.

5.4 Docking Periods

As stated in the Introduction, an in-water examination will take considerably longer than a conventional docking. The number and type of incidents analysed in docking reports suggested that a selective survey might be made of four defined areas namely a detailed examination of the forward, bilge and stern areas and a general examination of the remaining hull. This should reduce the time for the survey and allow consideration to be given to a possible docking programme as shown in Table 3.

Year	Docking	Examination
1st	Guarantee docking in dry dock	Examine for any initial problems
3rd	In-water survey	as outlined
5th	Special Survey 'A' Docking	as done now
7th/8th	In-water survey	as outlined
10th	Special Survey 'B' Docking	as done now

TABLE 3

Docking schedule.

6. PROCEDURE

In normal circumstances the shipowner will contact the Society when considering an in-water survey and a preliminary meeting will be held about three months prior to the survey date. A second meeting at which the survey contractor will also attend will be held closer to the date of survey and a more detailed plan of the operation of the survey will then be considered.

6.1 Preliminary meeting

At this meeting it should be verified that,

(i) the vessel is suitable for survey and the information required for classification purposes may be obtained,

(ii) the site location is acceptable and if a new site is proposed, there is evidence to show that it is also suitable (*see* Appendix 2),

(iii) the contractor is recognised by the Committee. If a new contractor is selected the Society would have to satisfy itself that the contractor was capable of doing the job (*see* Appendix 1),

(iv) there will be a suitable system for navigating on the flat of bottom if the ship is not already marked.

(v) any blanked off sea valves be credited towards the special survey,

(vi) copies of the impressed current log sheets will be submitted to Head Office as soon as possible,

(vii) arrangements will be made by the shipowner for repairing impressed current anodes and their shields should they be found to be damaged,

(viii) consideration is given for the safety of divers working near suction and discharges. A line of communication between the contractor and engine/pump room should be established,

(ix) a detailed plan of the whole maintenance afloat will be prepared including surveying, cleaning and painting and ballasting operations.

6.2 Second meeting

Discussion at this meeting should include: —

(i) results of the analysis of the impressed current log sheets,

(ii) agreement on the means of navigation proposed by the contractor if the ship is not marked,

(iii) verification that the deck edges have been marked.

(iv) agreement on the number and location of sea inlets to be blanked off and sea valves to be examined,

(v) the method proposed by the contractor for determining set-up in plating, i.e. air release,

(vi) the arrangements for the mooring of the ship, i.e. alongside a quay or at anchor, etc., likely survey draught and trim,

(vii) agreement on a detailed programme of the whole operation,

(viii) agreement on line of communication between the engine-room and pump room for the safety of divers,

(ix) the need for any cutting or welding gear or temporary staging to be provided by the contractor.

This second meeting may be unnecessary if the contractor and his method of operation are known to the Society but while in-water surveys are being developed it is considered worthwhile.

6.3 Survey detail

It is suggested that the survey may be itemised as follows when using CCTV: —

(i) The rudder, skeg, propeller, stern frame, port bilge keel, bow area, starboard bilge keel, fastenings and gratings, and the anodes, shields and reference cells and the remaining shell plating should be surveyed by diver held TV with two-way communication to the surface,

(ii) the flat of bottom should be surveyed using a vehicle with a diver and communications available to check items requested by the Surveyor.

(iii) the clearance readings of the pintles and stern tube should be done by a competent diver,

(iv) sea chests may be blanked off as requested by the owner.

It has been possible to complete a survey in one day where blanking off the sea chests was not required. However, the ship is usually available for three or four days for cleaning and painting so the survey may be spread over this period depending on the planning of the overall operation.

6.4 **On site**

The vessel, which should ideally have little or no heel or trim which might allow air to collect in indents under the ship, should preferably be tank cleaned with the tanks gas free for inspection in case it is necessary for an internal examination as a result of damage found during the survey. Providing there is no serious damage to repair, the maintenance work to the hull will generally amount to cleaning, surveying and painting. The sides above the water line will be cleaned with high pressure water jetting from barges, which also carry fresh water for washing down. Anti-fouling may then be applied down to the water line from special paint barges using conventional spray techniques. The flat of bottom which does not generally foul-up to the same extent as the sides may be cleaned with a machine to remove slime and by divers with rotary brushes to remove areas covered with barnacles, i.e. the underside of the bilge keels.

Items that can be repaired during the survey period should be examined first to give time for organising any such repairs. These include anode shields and corrosion on the rudder. If corrosion of the plating and vertical weld in way on the nose plate of the rudder has been occurring this may be repaired by trimming the ship to expose the corroded areas. However, the most difficult survey item is the inspection of the fastenings and gratings as some pumping systems in the ship will be in operation. Close liaison with the chief engineer is therefore necessary as soon as possible to organise this inspection with any planned ballasting arrangements, etc.

It is necessary to establish a line of communication between the engineroom/pumproom and the contractor so that the engineers know when it is safe to pump and the contractor knows when it is safe to dive. The safety of the divers is the contractor's responsibility, not the Surveyor's, but it is as well for all concerned to understand the dangers involved and precautions necessary to avoid accidents. In this respect the impressed current system should be switched off as previously mentioned.

Consideration should be given to repairing any damaged electric shields of the anodes by trowelling on some epoxy paint. This would have to be done in air. The heeling and trimming of the ship for cleaning and painting would probably expose the anodes at some stage so it should be possible to effect a useful repair. Surveys to date of the flat of bottom have involved either a remote or diver propelled vehicle controlled from the surface. It has been found that a series of transverse runs from forward to aft along the length of the bottom is the easiest way to navigate and carry out this part of the survey. A line should be hung over the side of the ship at the forward end indicating the frame number where it has been decided to start the flat of bottom survey. This is a reference point for the diver setting up the survey vehicle.

During the survey it may be decided to record certain parts of the inspection on video tape. This should be played back as soon as possible to establish that the machine is functioning correctly. It is suggested that video tape recordings be kept to a minimum and taken only when necessary. They are very time consuming to edit and require storage space. Video tape recorders are also very sensitive to the frequency used for recording. This creates a problem when moving between countries of 60 and 50 Hertz and when using a portable generator to provide power. The generator is usually being used for other equipment as well (lighting and T.V.) causing the load to vary. It has been found that a battery powered recorder, using rechargeable batteries, will provide tapes that may be played back successfully on a similar machine. When approving companies for in-water surveys, it is suggested that the Surveyor should be satisfied that such a recorder, or equivalent arrangement, is available otherwise the client might not be able to replay his tapes at his Head Office. The Society does not require video tape recordings of the survey unless there is a particular area that requires examination again.

A statement should be obtained from the Captain confirming that to his knowledge the ship has suffered no bottom damage since the last survey in drydock.

7. MAINTENANCE AFLOAT

7.1 **Cleaning**

It was thought that marine growth would be mainly confined to the sides of large ships and because of the deep draughts and lack of light the bottom would remain relatively clean. This would require the removal of grass, weed and barnacles from the side and slime from the bottom. Recently, however, there have been isolated cases of *conchoderima aurita* goose barnacles requiring removal from the flat of bottom. Sixty tons were removed in dry dock from one VLCC by hand scraping, water lancing having no effect on this species. The methods available for cleaning the submerged area of the hull mainly employ devices using rotary brushes. The brush may be varied depending on the type and degree of cleaning required. High pressure water jetting is under investigation although the possibility of corrosion from high velocity salt water particles striking steel plate must be considered. The removal of growth using explosives is also being tried. The machines using rotary brush techniques are SCAMP, BRUSHBOAT, BRUSHKART and diver held devices.

7.1.1 SCAMP

The SCAMP (Fig. 7) is a doughnut shaped machine using three rotary brushes powered by an umbilical cable from the surface. An impeller in the centre keeps the vehicle against the hull and helps to remove debris. It may be used on the sides or flat of bottom and may also be remote controlled from the surface. Where visibility permits it is usually controlled by a diver. The machine and its equipment are easily transported.

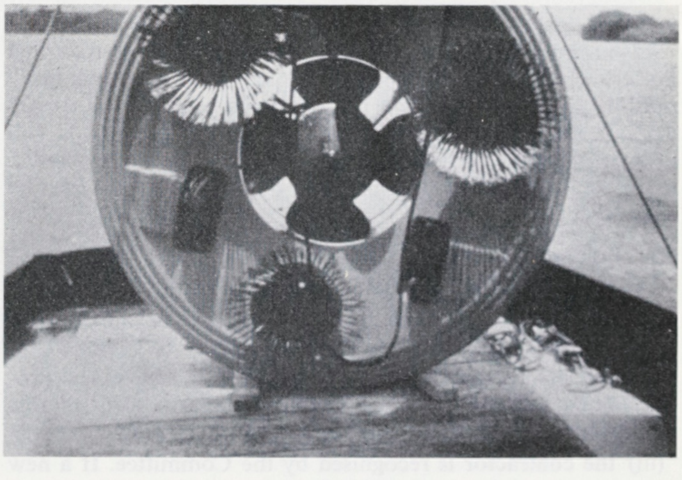

FIG. 7

SCAMP

7.1.2 BRUSHBOAT

The BRUSHBOAT (Fig. 8) consists of a launch having a 6·8 m long cylindrical brush suspended from the bow. When this is rotated in way of the hull a venturi effect is created and the entire unit adheres to the side of the ship. The speed of rotation of the brush may be varied to suit the degree of fouling. Cleaning is carried out at the rate of about 2500 m^2/hour. The brushboat has been designed for safe operation and is used at terminal ports. The whole unit is easily transported.

FIG. 8

BRUSHBOAT

7.1.3 BRUSHKART

The BRUSHKART (Fig. 9) is a portable diver operated cleaning machine which has been designed to clean up to 2500 m^2/hour. Hydraulic power is supplied through an umbilical cable by a diesel driven power unit. The machine grips the hull with a suction of about 660 kg by three rotary brushes and is driven at constant speed by its driving wheels. Powerful lights are fitted to the front of this machine.

7.2 **Painting**

Many trials have been carried out on in-water painting techniques with varying degrees of success using spray systems and a method of injecting paint into a pad. It appears unlikely, however, that a practical method will be evolved to satisfactorily coat the in-water area of the hull of a large ship. The technique used at present, which so far has proved satisfactory, is to light ballast the ship and paint to the waterline. The vessel may be heeled and trimmed to expose more of the hull. Care has to be taken to ensure suctions for pumping or operating machinery do not emerge above the water line. An exercise on a 215 000 tonne dwt tanker indicated that at a draught of 4·4 m forward and 5·9 aft of the vessel could be satisfactorily heeled at 13° to expose the bilge keel. A 280 000 ton dwt OBO ship recently achieved a heel of 10° in Las Palmas giving a draught amidships of 1·20 m; the mean level draught before heeling was 6·5 m (Fig. 10).

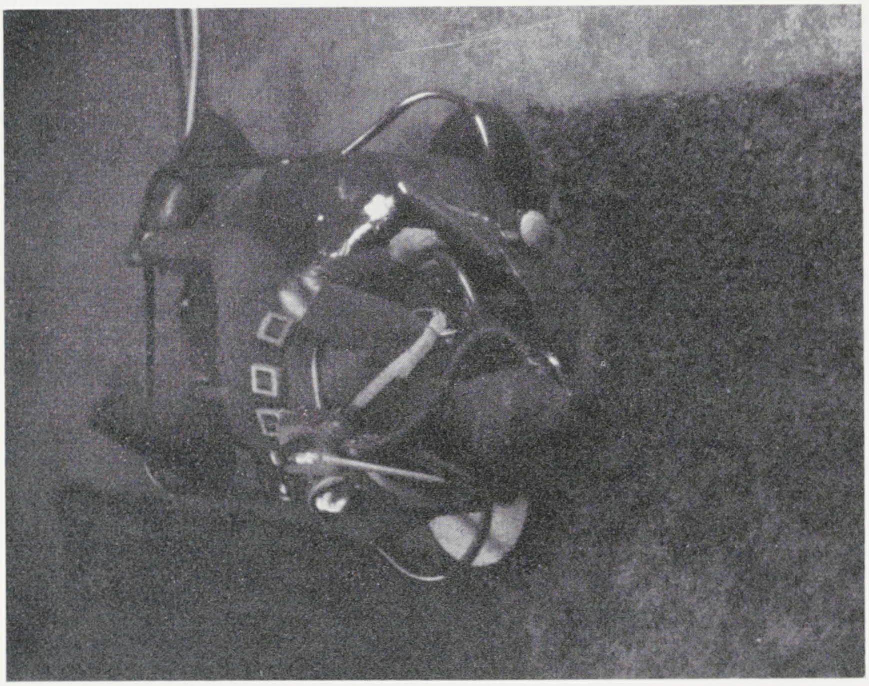

FIG. 9

BRUSHKART

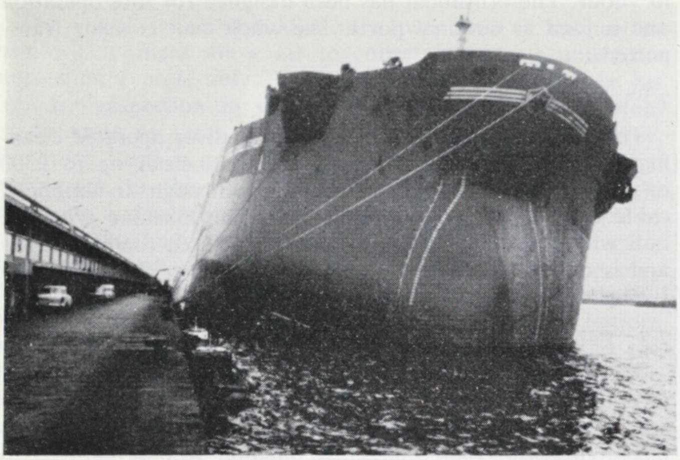

FIG. 10

Vessel heeled for painting.

8. CONCLUSIONS

Shipowners began requesting in-water surveys from July this year and at the time of writing this paper surveys have been carried out at Las Palmas and Marseilles. Limited experience was therefore available in helping to prepare the paper and to fully test the equipment and methods proposed for carrying out in-water surveys. It is hoped, however, that the basic method will remain satisfactory and that the suggestion for approving firms and checking out future sites will be of use to the local Surveyor.

The surveying, cleaning and painting afloat that has been achieved appears to confirm that there is a considerable cost saving to be made over normal drydocking practice. The afloat maintenance is estimated to cost less than 50 per cent of the same job in drydock and is producing savings in the region of £50 000 per vessel. It is thought that once shipowners have gained confidence in the heeling of their vessels and overcome the problems involved, i.e. boiler feed, exposed sea suctions, living at 10°–15° angles for short periods, etc., then cleaning and painting in air to the turn of the bilge will be a regular event.

If the flat of bottom of these large vessels should foul up to a significant extent it is likely that they will have to be drydocked to remove this growth and to apply anti-fouling. Las Palmas and Marseilles, where two companies have already been recognised by the Committee are likely to become established areas for maintenance afloat. Other areas that could develop are Malta, the Gulf area and possibly Sola in Norway. No mention has been made of the work involved with rigs and platforms in the North Sea but in-water surveys will be a definite requirement on these structures involving techniques considerably different from those being adopted for ships. Many platform structures are in a permanent position and will have to be maintained and surveyed at regular intervals. The maintaining afloat of ships, however, may or may not become a permanent feature depending upon the nature and extent of fouling on the flat of bottom and the subsequent techniques developed to clean this area.

ACKNOWLEDGEMENTS

The Author wishes to acknowledge the contribution made by Mr J. Huskins on impressed current cathodic protection systems and the permission of the following companies for use of photographs: J. G. Edmiston & Co. Ltd., Phocéenne Sous-Marine S.A., and Underwater Maintenance Co. Ltd.

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APPENDIX 1

APPROVAL OF FIRMS FOR IN-WATER SURVEYS

As laid down in Section 3 of the Rules the firms involved are to be approved for in-water surveys of ships. The Surveyor is to assess the technical and managerial ability of the firm to carry out a survey and is to submit his report to Head Office. The following has been drawn up as a guide to the Surveyor in his assessment of the organisation.

Guide lines for surveyors in assessing a company

Report on a firm desiring recognition by the Committee of Lloyd's Register of Shipping as being competent to carry out work under the terms of the Society's Provisional Rules for Periodical In-Water Surveys of Large Ships.

NAME OF FIRM:

ADDRESS:

DATE OF APPLICATION:

SURVEY CARRIED OUT BETWEEN: ....AND...............

NUMBER AND DATE OF ISSUE OF PREVIOUS CERTIFICATE:

1. **General**

State whether the firm is a separate company engaged in in-water survey activities, or is a special department of a general ship repair or other company.

2. **Management Structure**

Give concise description of management structure to assistant foreman level, or equivalent, including both underwater operations personnel and shore based personnel engaged in specialist activities.

3. **Installations**

Give brief description of quay facilities, if any, including depth of water available and state if surveys are normally carried out alongside the quay. If surveys are normally carried out off-shore, state if any permanent mooring facilities are available and if these are in sheltered water. State what maintenance facilities are available for the company's craft, diving gear and electronic equipment and if log books are kept for any equipment. If the firm does not carry out maintenance on important items, such as television cameras, state what arrangements have been made for the maintenance of this equipment.

4. **Code of Practice**

State how divers received their training, i.e. service personnel or members trained from other organisations, and where possible what code of practice the company uses for the conduct of in-water operations, i.e. any government regulations for the country.

5. **Records**

State if systemmatic logs of in-water operations are kept and if the firm supplies complete reports of each survey, including copies of photographs.

6. **Equipment**

List various craft used in connection with in-water operations. State, where possible, if these are well maintained. List items of equipment giving makers' names and where relevant state if adequate spare parts are available or readily available.

Equipment should be listed under the following headings:—

6.1 Surveys—all gear required to present survey information, i.e. TV cameras, lights, video tape recorders, vehicles, etc.

6.2 Other equipment—diving gear, any means for providing navigation on the hull, sea inlet and discharge blanks, etc.

6.3 Communications—make of diver/surface communication

6.4 Welding—mention any welding equipment available.

6.5 Cutting—mention any cutting equipment available.

6.6 Cleaning—as the ship must be cleaned for survey, the method of cleaning should be stated. If the company does not possess cleaning equipment state what arrangements are made for cleaning.

6.7 Painting—state if the company possesses equipment for painting afloat.

6.8 Non-destructive testing—state if the company possess any in-water NDT gear and give a description of the gear where applicable.

7. **Underwater Operations Personnel**

List all personnel (by rank but not necessarily by name) who may be engaged in in-water work. If certificates of competency in diving are needed, say in each case by what organisation or authority these are issued. By use of code letters against each member of the in-water operation personnel indicate competency to carry out various activities as follows:—

A—knowledge of ship construction, hull survey and repair work.

B—capable of taking poker and feeler clearance gauging.

C—television and still camera work.

D—non-destructive testing (state technique used).

8. **Tests**

If the survey of the firm has been carried out in conjunction with trials on a particular ship, a report on the trials and an assessment of results should be given. If all or any of the equipment is tested by simulated trials describe these trials and give assessment of results.

9. **Recommendation**

I/We, the undersigned Surveyor(s) to Lloyd's Register of Shipping, recommend to the Committee that Messrs.

....

(continue to) be recognised as a firm competent to carry out work in connection with surveys under the terms of the Society's Provisional Rules for Periodical In-Water Survey of Large Ships.

SIGNED ....

....

Surveyor(s) to Lloyd's Register of Shipping

DATE ....

APPENDIX 2

NOTES FOR ASSESSING A SITE

The site should satisfy the following conditions:—

(i) be sheltered from wind and swell and have a minimum tide,

(ii) be suitable for manœuvring a VLCC,

(iii) have good in-water visibility,

(iv) have good access for personnel and spares.

The following table should be filled in. This is similar to Table 1 of 4.4.1.

1	2	3	4	5
Water Surface	Ambient Light	Surface Incident	Disc Depth—Vertical	
			A	B

GUIDE:—

(1) For: Col. 1 use still or choppy.

Col. 2 use sunny or cloudy.

Col. 3 note the make of light meter as well as reading.

Cols. 4 and 5 the readings should be taken using a face mask.

(2) The readings A and B should be taken using a white disc, i.e. an enamel plate, attached to a graduated line. The reading to record is the depth at which the plate just disappears from view.

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Lloyd's Register Technical Association

A REVIEW OF CRACK AND FRACTURE MECHANISMS

C. Scholey and R. R. Lintell-Smith

Paper No. 3. Session 1975-76

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Hon. Sec. A. Wardle
71, Fenchurch Street, London, EC3M 4BS

A REVIEW OF CRACK AND FRACTURE MECHANISMS

By C. SCHOLEY* and R. R. LINTELL-SMITH*

TABLE OF CONTENTS

1 INTRODUCTION

It is a sad fact that despite all advances in engineering technology and design, service failures involving fractures in metals are still with us. The work of the Society's Research Laboratory over the past 20 years has shown that apart from a diminution in brittle fractures of ships' shell and deck plating there has been no decrease in other types of failures. Those most common include failures to engine components, shafting, propellers, tubes and pipes in very similar numbers to those of 1955.

The wide variety of failures investigated over the years has led to the acquisition of more and more sophisticated techniques for examining them and as a consequence the Laboratory has established a unique experience in the field of service fractures. It is worth mentioning that this experience is not confined to the marine field only but covers the complete spectrum of the Society's work in the world today. The ever increasing cost involved in the repair of machinery and power plant installations when coupled with the loss of revenue through 'out of service' time has resulted in the need for improved reliability. It is necessary therefore to obtain as much information as possible from any failure investigation in order that steps can be taken to prevent such breakdowns occurring in the future. More important than financial considerations however are those involving personal safety and many investigations have been carried out on failed components where loss of life or serious injury has been involved.

In recent years Surveyors attending the Society's Training Centre have shown a keen interest in the work of the Research Laboratory and in particular in the failure investigation work. Whilst discussing the appearance of cracks and fractures with the visiting Surveyors, the Authors have become aware of some confusion relating to the various modes of fracture and the terminology employed. It was considered appropriate therefore that in the twenty-first year of the Metallurgical Laboratory a paper should be presented to the Technical Association which would describe some of the more common fracture mechanisms experienced in engineering components, not only in service, but also during fabrication and new building.

These fracture mechanisms have been related to interesting investigations carried out in the Laboratory and it is the purpose of this paper not only to present a picture of the latest techniques in fracture examination and evaluation but also to convey some of the useful knowledge amassed by the Laboratory to Surveyors who are likely to meet with failed and fractured materials in their everyday work.

2 FRACTURE—GENERAL CONSIDERATIONS

Fracture can be considered to be a two stage process consisting of crack *initiation* and crack *propagation*. In the analysis of service fractures it is important to identify each of these components as it is possible for more than one mechanism to be involved in the failure.

At this stage in the Paper perhaps it is relevant to explain the essential difference between 'a crack' and 'a fracture'. At Crawley the word 'fracture' is favoured to describe a crack which has completely broken the component into two or more pieces. This may have occurred in service or during a mechanical test or may have been induced in the Laboratory to expose the faces of the crack. A 'crack' is regarded as a discontinuity which may or may not ultimately lead to fracture.

In general it is possible to classify fractures into two categories, namely, those involving appreciable plastic deformation prior to and during crack propagation and those exhibiting negligible or no gross deformation on fracture. The use of the words *ductile* and *brittle* in these two categories has purposely been avoided to minimise confusion, as these words generally describe a specific mode of fracture. For example, the second category is not restricted to the brittle fracture of welded mild steel structures with which we are all familiar, but includes the majority of other service failures.

Fracture occurs in many different ways depending upon the material and its condition, the state of stress and its rate of

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application, the temperature and the environment. Each mode of fracture is characterised by either its fractographic or its metallographic appearance. The term 'fractographic' refers to the service topography of a fracture and 'metallographic' appearance refers to its microstructural characteristics.

In some literature it has been suggested that cracks may be classified according to their 'intergranular' or 'transgranular' nature. This method of classification can be misleading as it is dangerous to assume that any one mode of fracture is wholly intergranular or transgranular as will become evident during this paper. Intergranular cracking as the term implies takes place along the grain boundaries and invariably gives rise to fracture surfaces exhibiting non-ductile features. Transgranular cracking occurs across the grains and may be accompanied by either appreciable plastic deformation or no deformation at all.

3 DUCTILE AND BRITTLE FRACTURE

3.1 Ductile

In the classic case of the fracture of most ductile commercial metals and alloys, as experienced by a tensile test at room temperature, the following sequence of events takes place.

Beyond the elastic limit, plastic deformation and strain hardening occur and at the point of maximum load the increase in strain hardening fails to compensate for the decrease in cross sectional area due to necking. A state of triaxial stress is developed in the necked region and fine cavities form around non-metallic inclusions and intermetallic particles. These cavities grow and coalesce to form a central crack which propagates perpendicular to the applied stress. Approaching the surface the crack grows along shear planes at 45° to the applied stress to form the cone part of the fracture. These steps are illustrated in Fig. 1. Visually the central 'cup' region of the fracture exhibits a fibrous or amorphous appearance. The shear lips or cone surfaces exhibit a similar appearance with the suggestion that a sliding action has taken place. Such ductile fracture is accompanied by considerable expenditure of energy in the form of heat which can be sensed by feeling the ends of a tensile specimen immediately after fracture.

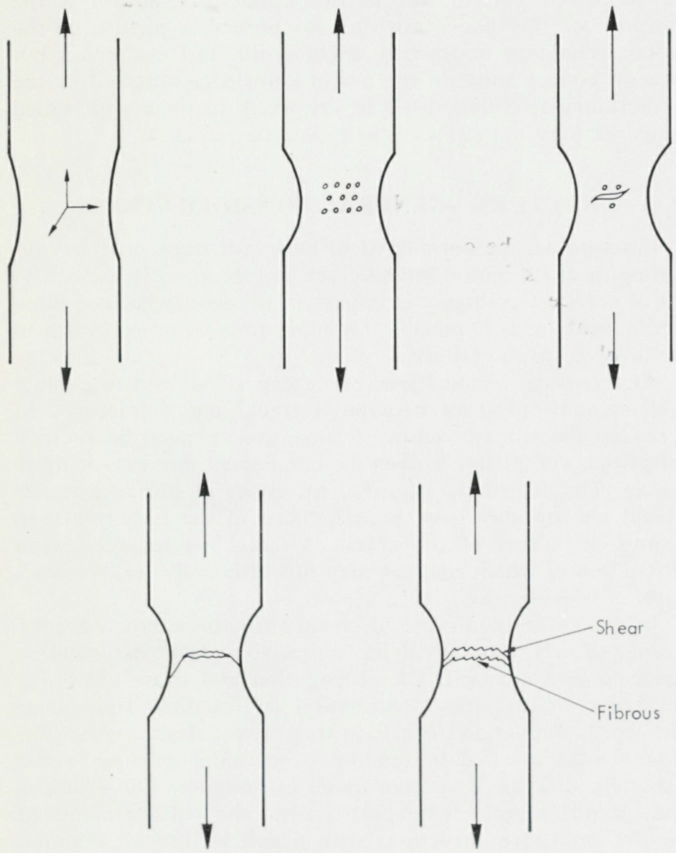

FIG. 1

Stages in the formation of cup and cone fracture.

The Scanning Electron Microscope (S.E.M.) installed in the Crawley Laboratory allows high power examination of fracture surfaces and the 'cup' region of a mild steel tensile specimen readily illustrates the void coalescence theory of ductile fracture as shown in Fig. 2. The fracture surface consists entirely of voids formed around non-metallic inclusions and is therefore called 'micro-void coalescence'. The shear lips or 'cone' part of the fracture exhibits similar but more elongated features.

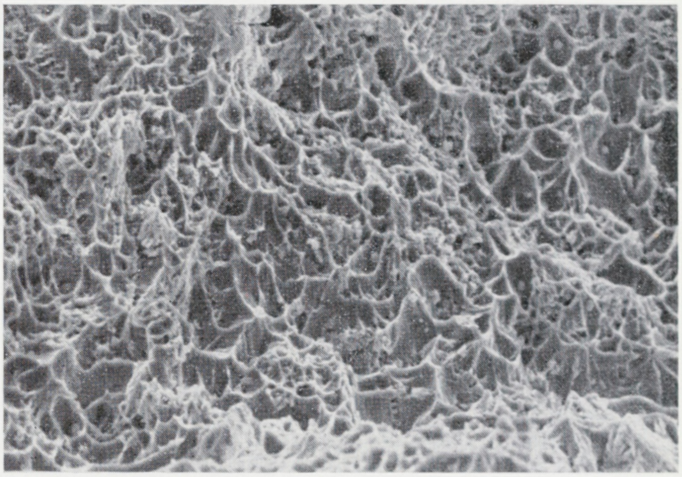

FIG. 2

Micro-void coalescence or ductile type of fracture in mild steel. (×1000)

Primary failure of a component in service by a ductile fracture mechanism can only occur under conditions of gross overstress. In practice such a fracture is usually detected at the tip of a crack formed by another mechanism, and where the remaining ligament has been unable to withstand the applied load. It should be noted that where more than one component has fractured, for example, during a machinery failure, it is usually found that those fractures involving gross deformation and necking are secondary fractures. The fracture which has not been accompanied by any appreciable deformation is probably the one of prime interest to the investigator. This has been found to be particularly true for bottom end bolt failures.

3.2 Lamellar tearing

Lamellar tearing can be regarded as a special case of ductile tearing. The tearing occurs beneath welds and in the 'through thickness' or 'short transverse' direction of rolled steel plate. It is always confined to the parent plate and can occur either

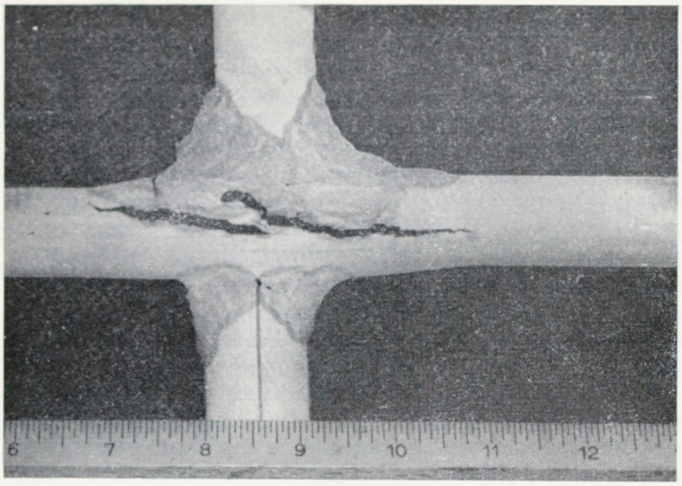

FIG. 3

Lamellar tearing in cruciform joint.

within or outside the visible heat affected zone. The tear generally runs parallel to the weld fusion line and in the same plane as the plate surface. It has been experienced predominantly in multi-pass full penetration tee butt joints and cruciform joints where strains, partially induced by welding, act through the joint and across the plate thickness. One notable case investigated in the Crawley Laboratory occurred in the cruciform joint between double bottom tank top plate, and bulkhead stool member (Fig. 3).

The presence of planar non-metallic inclusions in the rolled plate plays an important role in lamellar tearing. In cross section the tears are 'steplike' in nature and when broken open the exposed fracture surfaces exhibit a terraced appearance. High power examination on the S.E.M. shows that the fracture is essentially of a ductile type and consists of flat plateaus of coalesced voids around planar non-metallic inclusions, interconnected by shear walls. Thus the mechanism of lamellar tearing consists essentially of tearing open the matrix surrounding elongated non-metallic inclusions followed by shearing of the ligament between each 'torn open' group of inclusions. For lamellar tearing to occur therefore, sufficient strain must be developed in the short transverse direction to exceed the ductility of the plate across the thickness. In addition to thermally induced strains from weld metal shrinkage, strains imposed by the rigidity of the structure contribute to tearing. A relationship between short transverse direction ductility, non-metallic inclusion population and susceptibility to lamellar tearing has been established; those steels exhibiting low values of reduction of area in the short transverse direction have been found to be prone to lamellar tearing. As a consequence the short transverse tensile test is now included in material specifications of steels for critical applications. The recently issued British Standard Specification BS.5135 contains an appendix on lamellar tearing and gives details of joint designs where tearing may be expected. The appendix also contains some practical recommendations for its avoidance.

3.3 **Cleavage**

The other principal mode of fracture which can occur under a single application of uniaxial tensile stress is cleavage or true brittle fracture which occurs with no gross deformation. Cleavage is essentially fracture through grains and along planes predominantly orientated perpendicular to the applied stress. Visually such fractures appear bright and shiny and possess a crystalline or facetted appearance due to the reflection of light from the cleaved grains. At high power, cleavage fractures contain a large number of steps and exhibit a 'river' pattern of branching cracks (Fig. 4). Brittle fractures in plate material in particular are characterised by their visual appearance which consists of 'chevron' markings, as shown in Fig. 5; the 'chevrons' point to the origin of the fracture. Brittle fractures can be particularly insidious, occurring without warning, at low applied stresses and with a fast rate of crack propagation over considerable distances. Experience has shown that they can be catastrophic. The failure of welded Liberty ships and T2 tankers in the 1940's is well documented.

The occurrence of brittle fractures has not been limited to ship's plate however, other welded structures such as oil storage tanks, pressure vessels, pipelines, drilling rigs, bridges and chain cable have all suffered this form of fracture. Such

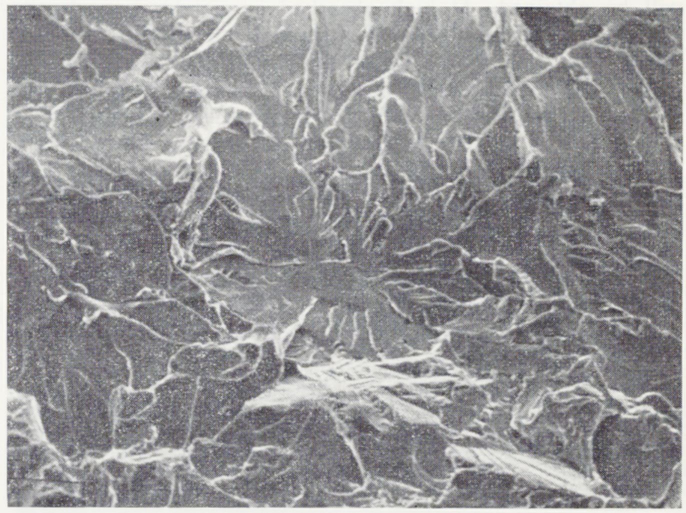

FIG. 4

Cleavage fracture in mild steel. (×1000)

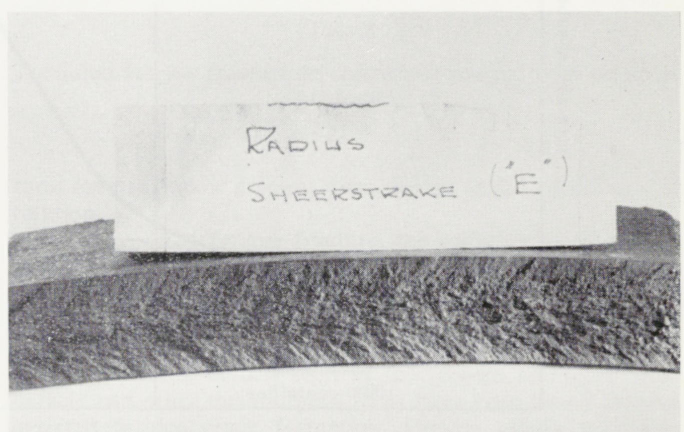

FIG. 5

Chevron markings on brittle fracture in ship plate.

calamities drew attention to the fact that a normally ductile mild steel could become brittle under certain circumstances. Investigations carried out on the failures sustained by all-welded ships showed that each fracture was due to a combination of a crack in a field of residual tensile stress in association with a steel that was notch brittle at the temperature when the casualty occurred. The investigations also showed that the probability of brittle fracture was very low if the minimum Charpy V-notch impact energy of the steel was 47J (35 ft/lbs) at 0°C and the fracture surfaces of the test pieces displayed less than 70 per cent crystallinity. As a consequence of these findings and the inclusion of the minimum C_V energy requirement in the Society's Rules for grade 'D' plate, the problem of brittle fracture has largely been contained.

The change in behaviour from normally ductile to brittle fracture with decreasing temperature is called the ductile to brittle transition and can be demonstrated by carrying out a series of Charpy V-notch impact tests over a range of temperature. A transition curve similar to that shown in Fig. 6 will be obtained. At high absorbed energy levels a ductile fibrous type of fracture with shear lips and plastic deformation at the notch root will result. At lower temperatures and with decreasing absorbed energy, areas of crystallinity or cleavage will appear in the fractures. The proportion of crystallinity will increase with decreasing temperature until, at the minimum absorbed energy level or lower shelf, the fracture becomes completely crystalline with no shear lips or notch root deformation. Steels which exhibit a very small transition temperature range are particularly dangerous as a small drop in temperature will result in the complete transition from ductile to brittle behaviour.

Unfortunately, the Charpy V-notch impact test does not provide test results which can be directly used to determine a safe minimum working temperature. The relationship between charpy V impact energy and service performance is of an empirical nature. In addition to those conditions previously stated susceptibility of steel to brittle fracture is increased by many factors, including the following:—

1. The existence of a state of a triaxial stress (as at the tip of a crack or sharp defect).
2. Decreasing temperature.
3. Increasing strain rate.
4. Increasing section thickness.
5. Unfavourable microstructure.
6. Increasing tensile strength.

The limitations of the Charpy V-notch impact test has resulted in the development of more sophisticated and expensive testing techniques in which a number of the above factors may be varied, for example, the Wells Wide Plate test, Plain Strain Fracture Toughness (K_{IC}) and Crack Opening Displacement tests. None of the tests so far devised, however, will produce the same transition temperature curve for a given steel.

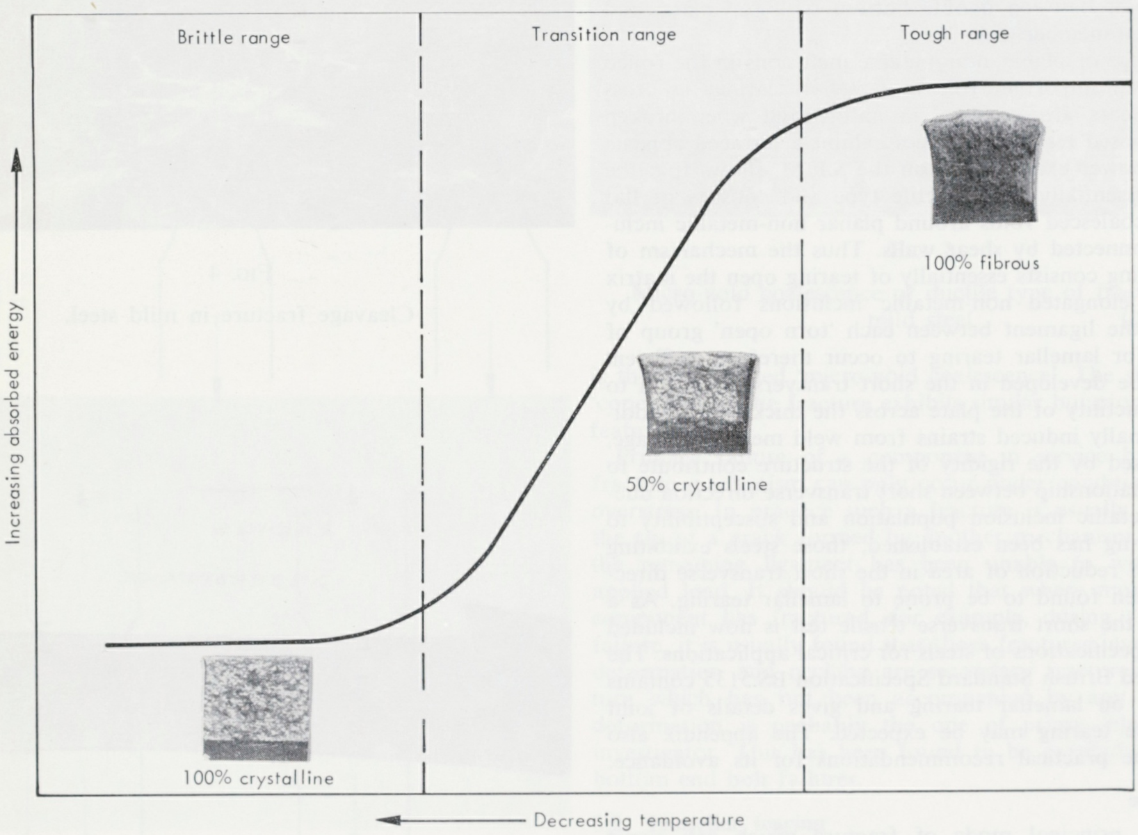

FIG. 6

Typical ductile to brittle transition curve.

In addition to the tendency for welding to introduce flaws into a structure, it also reduces toughness and increases the transition temperature of the parent plate material in the heat affected zones of the weld. In the heat affected zones of the weld the parent material is heated to a high temperature and rapidly cooled and the resulting microstructure is not that of the parent plate, therefore a change in its resistance to brittle fracture is to be expected. For higher tensile constructional steels and for low alloy steels the requirement for carrying out a post welding heat treatment operation is not solely aimed at stress relieving the structure. It must be emphasized that such treatments temper the microstructures produced in the heat affected zones of the weld and are effective in restoring the level of toughness in the heat affected zones towards that of the original plate material.

In concluding this section of the paper some mention should be made of the fracture characteristics of flake graphite cast irons. The crystalline fractures and nil ductility exhibited by broken tensile test pieces for example, are characteristics of cleavage or brittle fractures. This inherent 'brittleness' of grey cast irons under tensile loading is due to the incipient state of triaxial stress present at tips of the graphite flakes. This predominant tendency to fracture in a brittle manner renders the interpretation of fractures in cast iron particularly difficult.

4 FATIGUE FRACTURE

4.1 General

Of all failures investigated at the Laboratory, those involving metal fatigue are by far the most common. Fatigue fractures have been observed in failed samples from the entire range of the Society's work. They have occurred in components great and small and in both ferrous and non-ferrous metals. Metal fatigue may be defined as the characteristic failure of metals under the action of cyclic stress (as distinguished from failure under steady stress). It is a well established fact that metals can be made to fail under the action of cyclic stress even though the magnitude of that stress is well below the static breaking stress. The recognition of the significance of the effect of fluctuating stresses on metals occurred relatively late in the history of engineering science and the phenomenon of fatigue failure is still under study. Historically, it's discovery is generally attributed to railway engineers of the early 19th century as compared with early studies of tensile fractures in metals believed to have been made by Leonardo da Vinci.

The cyclic stresses involved in the formation of fatigue cracks and fractures are basically of three types: fluctuating direct stress, fluctuating bending stress and fluctuating torsional stress. A fourth type, fluctuating thermal stress will be dealt with later. All fatigue fractures produced by these stresses have certain common characteristics, the first of which is that, with the exception of rare cases, the fractures are transcrystalline in character, non-ductile and occur without gross plastic deformation. Another common feature is that the fracture appearances, whether produced by bending, torsional or direct cyclic stress, are similar. This similarity is illustrated in the crankshaft fractures depicted in Figs. 7 and 8 which show typical bending and torsional fatigue fractures respectively. The two types of fracture differ only in their orientation with respect to the applied cyclic stress. In the majority of cases, bending and direct stress fatigue fractures are normal to the plane of the stress. Torsional fatigue fractures tend to follow a spiral form angled at 45° to the axis of rotation.

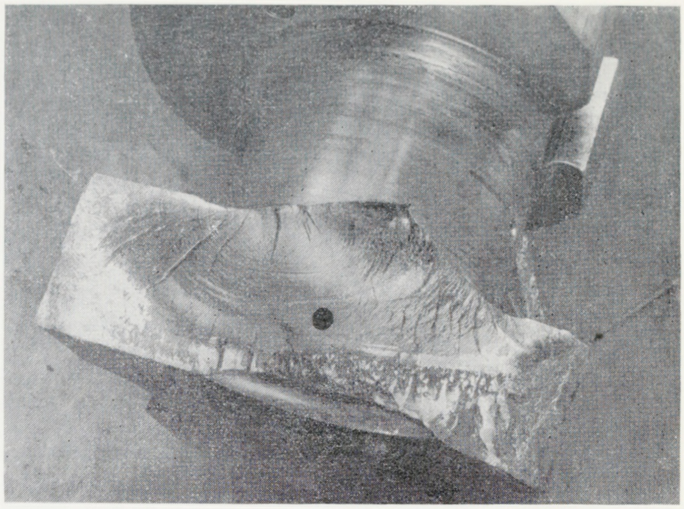

FIG. 7

Bending fatigue fracture in web of crankshaft.

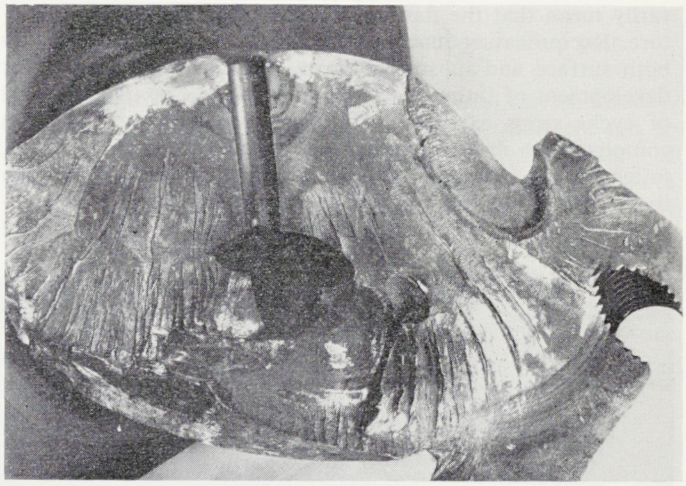

FIG. 8

Torsional fatigue fracture in crankshaft journal with origin at oil hole.

In all types of fatigue fracture, the initiation of the crack is greatly influenced by the presence of local stress raisers in the component or structure. These stress raisers may include fillet radii in shafts and other components, the roots of threads in bolts, the toes of welds and features such as oil holes in journals. Casting defects such as that shown in Fig. 9, weld defects and other metallurgical flaws have been found to have initiated fatigue crack formation. Fatigue cracks may also have their origins in corroded or pitted areas of reciprocating parts particularly where fretting (see later) is present. All the foregoing flaws, defects and surface imperfections are themselves stress raisers.

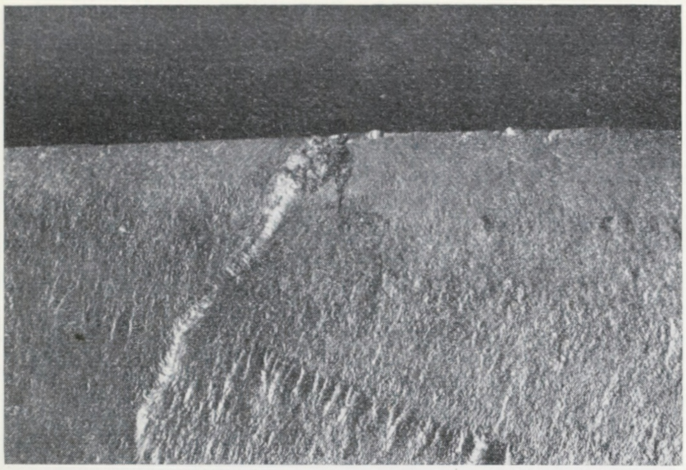

FIG. 9

Casting defect at origin of fatigue fracture.

It is important to realise, however, that the detection of such a flaw at the origin of a fatigue fracture does not necessarily mean that the flaw in itself is the prime cause of fracture. Reciprocating machinery parts have run for years with both surface and subsurface flaws in them. The criterion for development of fatigue cracking must always be that the level of cyclic stress exceeds the fatigue strength of the material complete with its flaws and stress raisers. In service fractures, fatigue crack propagation may proceed completely through the section of the component or until the component is sufficiently weakened as to cause the remaining ligament of metal to rupture. Fracture appearances vary, therefore, from the nearly 'all fatigue' type to those which are made up of part fatigue and part ductile or brittle fracture.

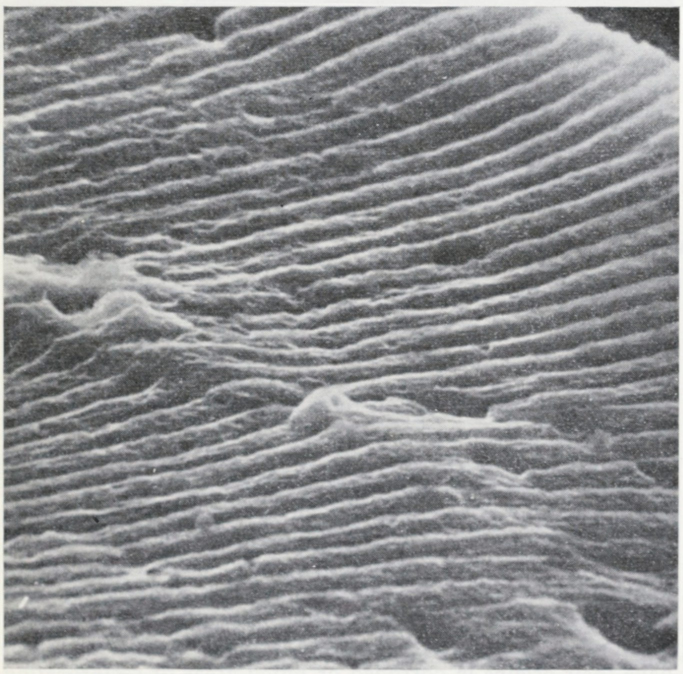

FIG. 10

Fatigue striations on fracture surface. (×6000)

Fatigue crack propagation in metals is understood to proceed by a striation mechanism, producing one growth increment per loading cycle. The true nature of the striations may only be appreciated by the use of a Scanning Electron Microscope to observe the fracture. A fractograph of a typical fatigue fracture taken on the Laboratory's instrument is shown in Fig. 10. Visually, however, the fracture is characterised by 'beach' markings on the surface which radiate from the locus or origin and are generally at right angles to the direction of crack propagation. These beach markings are 'arrests' in the development of the fracture and are due to the non-uniform occurrence of the maximum stress. Stepped ridges extending in the direction of crack propagation are often present on fatigue fractures and are commonly associated with fracture origins (Fig. 11).

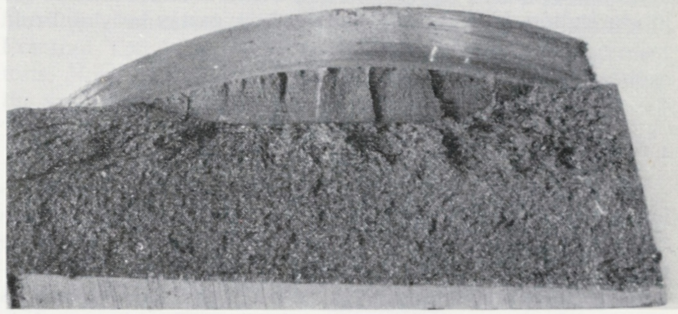

FIG. 11

Stepped ridges associated with multiple origins of an exposed fatigue crack.

The distribution of the striations on the surfaces of fatigue fractures is understood to be a function of the frequency and amplitude of the cyclic stresses producing the fracture. Higher stresses produce wider spaces between the striations and, by definition, the number of striations is directly related to the number of stress reversals to cause failure. As a consequence investigators are frequently asked to determine the age of the fatigue fracture in question, and the number of stress cycles to cause failure. Moreover, a great deal of emphasis is placed upon the degree of smoothness or coarseness of the fracture in determining its age. In the Authors' experience, however, smooth polished fractures have been exhibited by cracks of both short and long propagation times although fatigue fractures showing coarse features are generally assumed to be rapid and hence of short duration. Numerous factors combine to render interpretation difficult, not the least of which is the degree of movement and fretting that has occurred between the crack faces.

In general, therefore, the only factual diagnosis obtainable from a visual examination is that the observer may recognise a fatigue fracture and its origins by the characteristic appearance. Without accurate knowledge of the service life and conditions of the affected part, any assessment of the stress range and time to failure is highly speculative. The scanning electron

microscope has greatly increased the Laboratory's facility for confirming the presence of a fatigue fracture. Background information is required, however, to enable more precise and quantitative observations to be made.

One type of fatigue fracture which has come to be associated with higher cyclic stress ranges is that in which the fracture is characterised by the development of multiple stepped ridges extending in the direction of crack propagation (Fig. 12). In a component of circular section, such as the one illustrated, the stepped ridges appear as radial lines on the fracture. The characteristic 'beach' markings are often faint and sometimes absent from such fractures. The fracture faces are frequently coarse and almost brittle in appearance and their fatigue nature is only confirmed by close scrutiny under the Scanning Electron Microscope.

In the ideal case of the single origin fatigue fracture, the only markings on the fatigue fracture surface are those radiating outwards from the origin and the origin is easily pinpointed as the locus of the 'beach' markings. In many cases, however, fatigue cracking may initiate at a number of origins and the final fracture face may therefore contain several intersecting fatigue cracks each propagating from its own origin. A typical multiple origin fatigue fracture of this type which occurred in a copper alloy bolt is shown in Fig. 13. The higher stress type of fracture involving the formation of radial stepped ridges (*see* Fig. 12) is also a form of multiple origin fracture.

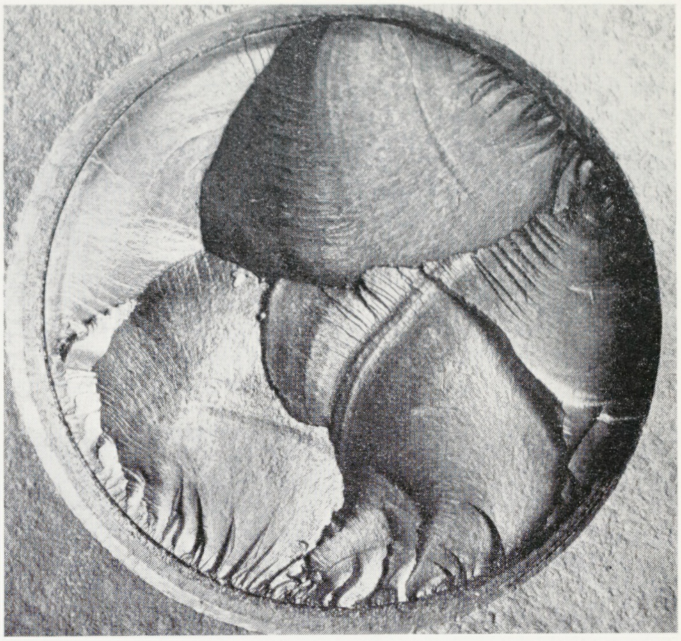

FIG. 13

Multiple origin fatigue fracture in copper alloy bolt.

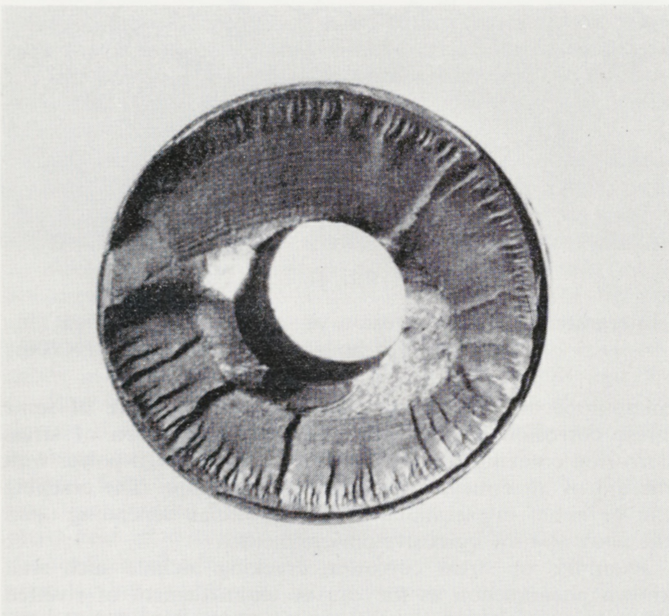

FIG. 12

Multiple origin, higher cyclic stress fatigue fracture.

In the preceding general remarks on fatigue fracture, reference has been made to rare cases of fractures which are not transgranular. The Laboratory has little experience of the phenomenon but it is known that fatigue failure can adopt an intergranular mode where grain boundary weakness of the material is present. A suspected case of such a fracture, in the nitrided skin of a gear, has been encountered at the Laboratory. Research work on the subject is still in its infancy, however, and the incidence of failures is small enough to warrant little attention at the present time. This mode of fracture may well assume a greater importance in the future if, for example, the loading on case hardened gear teeth is significantly increased.

4.2 Corrosion fatigue

Corrosion fatigue, of which fretting fatigue may be said to be a variety, is simply defined as the behaviour of metals when subjected to cyclic stress and simultaneously exposed to a corrosive environment. The combined effects of these two influences greatly exceeds their individual effects. Consequently materials which exhibit a fatigue limit when tested in air at room temperature show no fatigue limit when tested in a corrosive environment. Moreover, when testing such materials in air the tests are not affected by the speed of testing over a range from about 1000 to 12 000 cycles/min. When tests are made in a corrosive environment there is a marked dependence on speed. Since corrosion attack is a time-dependent phenomenon, the higher the testing speed, the smaller the damage due to corrosion. Hence, at lower frequencies of cyclic stress, the number of cycles to failure is lower, though total time to failure may increase.

The occurrence of failures due to corrosion fatigue is relatively high but, with the exception of fretting fatigue which is a surface contact phenomenon, it is rare for the opened-up fractures to have escaped corrosion attack. Fracture appearances may vary, therefore, from the classic fatigue type with shallow corrosion attack at the origin to a completely oxidised fracture from which the cause is inferred rather than proved. An example of corrosion fatigue where gross corrosion attack had occurred was recently examined. The failure involved the collapse of a Scotch boiler furnace tube in which circumferential corrosion fatigue cracks developed from the water side surface. A photograph of the

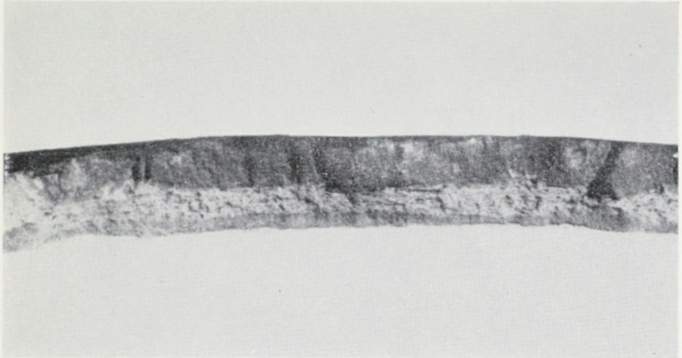

FIG. 14

Heavily corroded circumferential corrosion fatigue in a fractured furnace tube.

exposed fracture surfaces is shown in Fig. 14. The appearance of the fracture shows that the fatigue crack had become coated with black magnetite and possessed a fairly uniform crack front. It had extended more than halfway through the tube wall thickness before final rupture of the remaining ligament.

4.3 Thermal fatigue

Thermal fatigue, a phenomenon involving alternating thermal stresses, is rarely seen in its purest form in the failures coming to the Laboratory from the Society's clients. The service conditions producing the majority of thermal fatigue failures examined have involved corrosion or oxidation and the failures have been principally to steam tubes, pipes and valves. The cracking can be of two types, unidirectional cracking and 'craze' cracking. Where unidirectional cracking of tubular components is involved, the cracks formed are similar to corrosion fatigue cracks. Craze cracking, however, is a surface phenomenon where the skin of a metal is thermally cycled. Such conditions are generally attributed to water droplets impinging on the heated metal surface. The cracks so formed may ultimately penetrate the complete wall thickness of the component. An example of such cracking (as revealed by magnetic crack detection) in the bore of a main steam valve is shown in Fig. 15.

FIG. 15

Craze cracking in bore of main steam valve.

5 STRESS CORROSION CRACKING

Stress corrosion cracking has been defined as the spontaneous failure of metals by cracking under the combined action of corrosion and steady tensile stress, whether residual or applied. Stress corrosion cracking should not be confused with corrosion fatigue cracking which occurs under conditions of corrosion and cyclic stressing. However, the common factor necessary for both stress corrosion and corrosion fatigue cracking, namely corrosion, does sometimes lead to a similarity in the appearance of both forms of cracking and makes interpretation difficult. The deposition of a metal oxide on the crack faces is an unavoidable part of the corrosion process and this oxide coating can destroy the fractographic details. In certain cases where the oxide coating has been thin, however, it has been possible to resolve on the scanning electron microscope at high power the intergranular nature of some stress corrosion cracks. The positive identification of stress corrosion cracking can only be carried out at high power with the aid of an optical metallurgical microscope. The cracking can be either intergranular or transgranular depending upon the alloy and the corrosive environment.

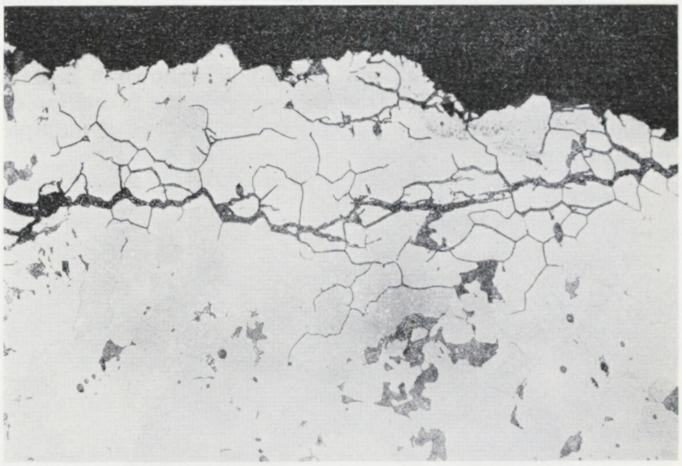

FIG. 16

Intergranular stress corrosion or 'caustic embrittlement' in riveted boiler drum. (×200)

Examples of stress corrosion cracking include such well known phenomenon as the caustic embrittlement of rivetted boiler drums and the season cracking of brass tubes. In the former case, alkali present in incorrectly treated boiler water is concentrated in the crevices under rivet heads which ultimately leads to the development of oxide filled intergranular cracks in the boiler drum steel (Fig. 16). Such cracking is less prevalent in welded boilers which are fairly crevice free and only one case of stress corrosion cracking in a welded boiler drum, not stress relief heat treated after welding, has been investigated at Crawley. The season cracking of brass tubes in ammoniacal atmospheres is also intergranular in nature and the stress component is residual from the cold drawing operation. The problem can be avoided by carrying out a low temperature stress relief or annealing heat treatment

which does not destroy the work-hardness of the tubes. In the Laboratory at Crawley, stress corrosion cracking has been detected in a wide variety of components, such as a low alloy steel H.P. steam turbine rotor, high tensile brass propellers, austenitic stainless steel autoclaves and pressure vessels and steel components from sulphur bearing oil wells. Fortunately there are no recorded cases of ship plate having suffered stress corrosion cracking in a marine environment. For low alloy and carbon steels the susceptibility to stress corrosion cracking can be expected to increase with increasing hardness. The critical level of hardness above which stress corrosion cracking will occur is dependent upon the level of stress and the corrosive environment. It is difficult therefore to specify a maximum level of hardness which can be tolerated, in the heat affected zone of a weld for example, without rendering the material susceptible to stress corrosion cracking.

The cases of stress corrosion cracking in manganese bronze (high tensile brass) propellers investigated in the Laboratory have invariably been found to be associated with the presence of microstructures consisting predominantly of *Beta* phase. It is the practice of propeller manufacturers to adjust the copper and zinc contents of the alloy to give a microstructure consisting of soft particles of *Alpha* phase in a matrix of harder *Beta* phase. An increase in *Alpha* content gives reduced strength, increased ductility and improved fatigue resistance; the optimum *Alpha* content is approximately 40 per cent. The *Beta* phase in these alloys is susceptible to stress corrosion cracking and a reduction in *Alpha* content gives rise to increased sensitivity. For this reason the Society's Rules for manganese bronze propellers now require that a sample from each tensile test piece, representative of a manganese bronze propeller casting, be metallographically examined and the proportion of *Alpha* phase determined. The proportion of *Alpha* phase is to be not less than 25 per cent.

Unfortunately, when manganese bronze alloys are heated to above 550°C the *Alpha* phase dissolves into the *Beta* to produce an entirely Beta phase microstructure. If the alloy is allowed to cool slowly to room temperature, reprecipitation of *Alpha* occurs and a structure similar to the original is obtained. If the alloy is rapidly cooled, however, the 100 per cent *Beta* phase microstructure can be retained at room temperature. Such a cycle of heating and rapid cooling is experienced in the heat affected zone of a repair weld and the resulting 'all *Beta*' structure and high level of induced residual stresses render the heat affected zones particularly susceptible to stress corrosion cracking if not post-weld stress relief heat treated. Such a case investigated in the Laboratory showed that the cracking was wholly intergranular in the 'all *Beta*' phase heat affected zone. Beyond the heat affected zone in the *Alpha*/*Beta* structure of the casting the cracks were transgranular but confined to the *Beta* phase. The presence of a predominantly *Beta* phase structure is not essential for the development of stress corrosion cracking if the stresses are sufficiently high. Mechanical damage suffered by the leading edges of manganese bronze propellers for example, can lead to the development of fine stress corrosion cracks which in turn can act as stress raisers for the initiation of fatigue cracks. The improved corrosion resistance of iron and nickel bearing aluminium bronzes renders these alloys insensitive to stress corrosion cracking and no cases of such alloys having developed stress corrosion cracking have been experienced by the Laboratory.

With regard to the stress corrosion cracking of austenitic stainless steels (18 per cent Cr/8 per cent Ni) in aqueous solutions containing chlorides, the predominant mode of cracking is transgranular (Fig. 17). The intergranular cracking of austenitic stainless steels which is often referred to as 'weld decay' is not strictly a stress corrosion process. The phenomenon is associated with the precipitation of chromium carbide at grain boundaries which results in the adjacent matrix becoming impoverished in chromium, the protective element. Subsequent exposure to a corrosive environment results in selective attacks of the areas adjacent to the grain boundaries and complete break up of the microstructure in an intergranular manner. The problem of intercrystalline corrosion attack and its prevention, together with details of tests for determining the susceptibility of austenitic stainless steels to weld decay, are adequately covered in *Instructions to Surveyors,* Part 8, 1971, Welding of Ships.

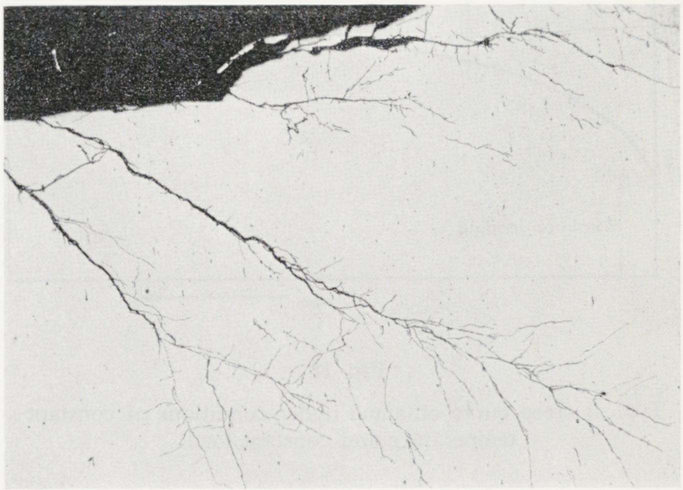

FIG. 17

Transgranular branching stress corrosion cracks in austenitic stainless steel. (×100)

6 FRACTURE AT ELEVATED TEMPERATURE

When a metal is subjected to a constant tensile load at an elevated temperature, it will undergo a time dependent increase in length and is said to have suffered 'creep'. The rate of increase in length or 'creep rate', however, is not linear. An idealised creep curve which would be obtained at constant temperature under conditions of constant load is shown in Fig. 18. This curve illustrates the three stages associated with creep fracture, namely a primary stage of decreasing creep rate, a secondary stage of nearly constant creep rate, and a final tertiary stage of increasing creep rate which precedes final fracture. Under creep conditions the metal will ultimately fail even though the load is below that which would have caused failure had it been progressively and rapidly applied. Moreover, the elongation at fracture of a metal suffering such a creep failure will be less than that produced by progressive and rapid loading.

The majority of the elevated temperature fractures examined in the Crawley Laboratory have been associated with tubular components from steam raising plant or petrochemical installations. In general the fractures can be divided

into two categories, namely

1. Those which are preceded by considerable 'ballooning' or bulging and extensive wall thinning prior to fracture (Fig. 19).
2. Those which may or may not be preceded by bulging and are not accompanied by pronounced wall thinning.

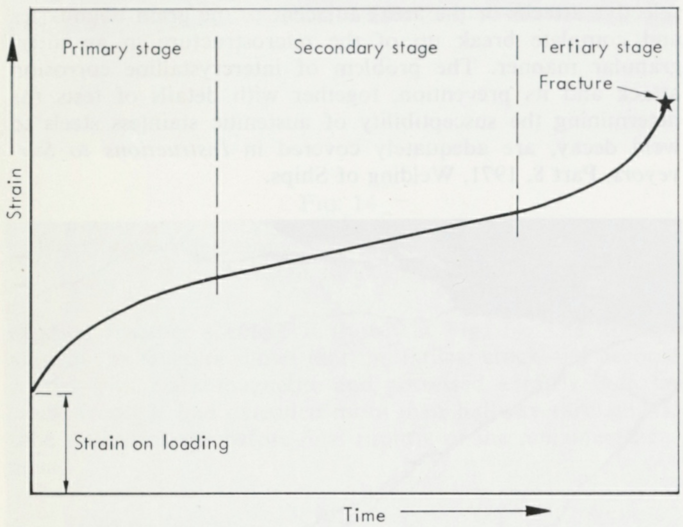

FIG. 18

Typical creep curve obtained under conditions of constant temperature and constant load.

The former 'ductile' type of burst is associated with a progressive rise in tube wall temperature with a corresponding fall in mechanical properties of the tube material. The tube wall is then unable to withstand the normal working pressure and bulging followed by bursting occurs. The resultant rapid drop in pressure and escape of steam and water effectively quenches the overheated tube material to produce a coarse grained and hard martensitic microstructure. This type of burst is considered to be either a high temperature rapid creep failure or can be likened to a high temperature tensile test type of failure. The failure illustrated probably occurred due to a fairly rapid increase in temperature of the tube wall, resulting from water starvation.

FIG. 19

Burst in heater tube accompanied by bulging and wall thinning.

The type of failure which is not accompanied by pronounced wall thinning can be regarded as a true creep fracture, where continual slow deformation occurred under constant stress and at a constant temperature in excess of the normal operating condition. Such fractures are usually found to be intergranular and are accompanied by extensive grain boundary cracking in the steel which may be evident in the form of surface cracking adjacent to the major fracture (Fig. 20). The failure of the grain boundaries is thought to be associated with the tertiary stage of the creep process. This type of fracture is not short term and could take a considerable period of time to develop, time being measured in years rather than minutes and hours. Similar to other fracture mechanisms the presence of a flaw in a weld or a sharp notch at the toe of a weld could act as a stress raiser for the initiation of a creep crack.

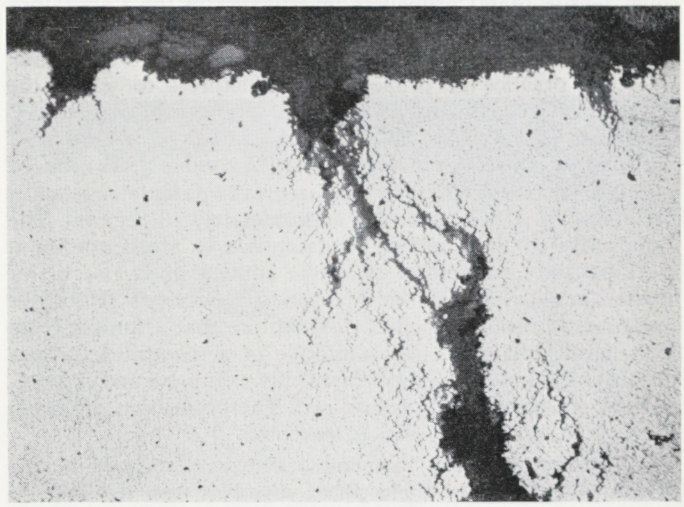

FIG. 20

Typical intergranular creep cracking. (×100)

7 COMMENTS

The aim of this paper has been to describe in a simple manner the principal modes of fracture by which metals can fail. The contents of the paper and the examples illustrated are entirely based on the experience gained by the Research Laboratory at Crawley. The paper is intended to be informative and of practical value rather than a theoretical treatise for the specialist and it must be realised that the subject cannot be fully covered in a paper of this nature.

Although the paper deals with fracture mechanisms the Authors are aware that the prime concern of the Society in this field must be the prevention of service failures. With this in mind therefore it is hoped that the paper will be of general benefit to Surveyors in the field and of particular assistance should they be faced with such an unwelcome event as a major breakdown.

The topics covered are considered to be those most relevant to the activities of the Society in both the marine and non-marine fields. Except for lamellar tearing, no mention of cracking due to welding has been made and in this respect it was considered that such cracking properly belonged in a paper devoted to welding and its problems.

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LLOYD'S REGISTER AND THE SHIPREPAIRER

T. Andersson
(Guest Speaker)

Paper No. 4. Session 1975-76

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LLOYD'S REGISTER AND THE SHIPREPAIRER

Read before the Technical Association by T. Andersson, Director and General Manager Götaverken Repair Division, the President, Mr. W. McClimont in the Chair.

Bang! The bottle breaks just after the godmother has said 'May happiness follow you in all your days', and the ship slides from the building berth into her natural element.

But despite all the good wishes, the cheers and the banners, everyone knows that there will not be unbroken happiness in the future of the ship. Both owners and classification societies know that there will be many testing times during her working life of maybe more than 25 years. She will meet heavy weather in the North Atlantic, long swells south-east of Durban or maybe a typhoon in the China Sea. However well designed the ship may be, she may still be involved in collisions, or catch fire, or run aground—all causing damage that will have to be repaired. Also, of course, the ship will undergo regular inspections and drydockings to insure that she maintains her seaworthiness and class. The major repairs a ship may require during her lifetime are undertaken by certain shipyards which have specialized in this kind of work, situated at various locations around the world. At these ship repair yards and many other places, the surveys required by Lloyd's Register will also be carried out.

In a previous paper presented to the Technical Association Mr. Ross Belch spoke on behalf of the shipbuilder. Much of what was said about personal relations in a shipyard, skilfulness of Surveyors, disputes between Surveyor and shipyard is also valid on the repairing side, and there is no reason for me to repeat what was said so brilliantly. Maybe life in a repair yard is more hectic, though, as cutting time is an all-important factor. A lot of responsibility rests with the local Surveyor. Theoretically disputes can arise, and in fact do arise, but in the Author's opinion confrontation problems in a shipyard arise from differences between individuals rather than conflicting interpretation of Rule requirements. The Author therefore considers this type of dispute, fortunately occurring rather seldom, of little interest and can find no reason for enlarging on this point.

In a repair yard a game between four parties is played, namely between the owners, the repairers, the underwriters and the classification society. They must find solutions acceptable to them all. Lloyd's Register of Shipping is the largest of the classification societies and has therefore very great influence. The classification society is the only impartial body in the game and is free from the commercial interests which influence the other parties. Its decision is law. The Author has no experience of surveyors having misused their power, and perhaps the Society's excellent system of having a 'second opinion' appeal not only helps the younger surveyor to reach a correct solution, but prevents confrontation and acts as a safety valve.

The nucleus of any ship repair yard is a facility for taking the ship out of the water—for small vessels, perhaps a slip or 'marine railway', and for larger vessels a graving dock or a floating dock. Such drydocks have been established in virtually all parts of the world where ships discharge and along the normal ballast routes. With increasing number and size of vessels, it has been difficult to keep pace with the demands of the market, especially as vessels require not only drydocking facilities but also many other services in the form of suitable moorings, engineering shops, trained workers, etc. In the past, repair facilities were usually included in shipyards engaged in new construction but more recently a number of specialized repair yards, intended particularly for the biggest tankers and bulk carriers, have been built.

Drydocking

During a ship's normal operation there will be a decrease in speed due to fouling. This can be minimized by using the increasingly sophisticated painting systems now available, which reduce the necessary frequency of drydocking. Not long ago, drydocking used to take place practically every year, whereas today more than two years' operation between dockings can be expected with a modern painting system combined with in-water cleaning. But even the best paint cannot prevent the necessity for drydocking sooner or later, and moreover the requirements of Lloyd's Register call for inspection in dock, especially of the rudder, tailshaft and sea valves. (Systems permitting in-water survey are currently being developed by Lloyd's Register together with some major tanker owners.) Due to shortage of time and difficulties with pumping around ballast water in a tanker resting in a dock or laying along the quayside, much of the routine classification work previously carried out in a drydock can, with great advantage, be dealt with before the ship arrives at the repair yard. Before entering the drydock the ship, if a tanker, will have to be gasfree, which normally is a rather laborious procedure. A modern drydock for tankers will therefore also have to have a tank-cleaning station in the vicinity. The new agreement between all the countries around the Baltic, which is recommended to come in force from the 1st of January, 1977, will ensure that all yards occupied in the repair of oil tankers must have a tankcleaning station connected to the activity.

Capital cost of the ship

In view of the increasing cost of ships, as well as the enormous cost of the drydock itself, everything must be done to shorten the time occupied by drydocking. A modern drydock will therefore be equipped with very efficient ship handling and positioning systems, and powerful pumps and other arrangements such that even the largest tanker can be docked and dry in about four hours. Refilling and docking-out the ship may require two to three hours. If cleaning and painting are carried on continuously, the total time required in the drydock will be less than 100 hours.

Special problems of very large vessels

The very large tankers and bulk carriers built in recent years have created new problems in connection with docking and the even larger ships which have been envisaged would surely aggravate these problems. One of the major problems connected with drydocking is the supporting of the ship by keel and bilge blocks. The ship generally enters the dock in ballast condition with 1,0–1,5 m trim by the stern. In most cases this means that the ship is in a pronounced hogging condition. The design of a typical VLCC usually prohibits support under the keel only—side support is also required.

When docking a large ship, it must be possible to continuously counteract the effect of wind and current. A minimum draught of 6 m may be required to handle ships with a moulded depth over 30 m, in order to prevent the ship from being over influenced by wind forces. Tugboats cannot normally enter the dock with the bigger ships. There would be insufficient space for the tug manœuvres necessary to achieve sufficient side force to prevent the ship from being damaged. However, shorebased ship-handling devices consisting of automatic mooring winches and railborne 'mules' on the dockside have proved very effective. If such devices are available, then a shore gang of seven men is enough for the safe handling of ships of any size, even with high wind and current forces acting on the ship.

As previously mentioned, a certain minimum draught is required for safe handling while entering the dock. Once inside, however, ballast must be reduced to the minimum in order to reduce the stresses on ship and dock when dry. The ballast should also be distributed to give a slight trim by the stern so that the ship will first touch aft lightly when the dock is emptied. In practice, a compromise is usually adopted to avoid extensive discharging of ballast after entering the dock and before the dock can be emptied. The deformation of the hull in various ballast conditions must also be considered, since certain ballast distributions cause excessive hogging and thus very high loads on the blocks forward and aft. This has indeed been observed in many cases.

When docking ships of well above 100 000 tonnes dwt it is no longer feasible to use the old system of allowing the ship to settle on keel blocks and pumping out the dock to a level of 1,0–1,8 m below the docking draught of the ship before a limited number of side blocks are manœuvred into position and the vessel shored. This would result in about 90–95 per cent of the loading coming on the keel blocks. In many cases, therefore, shipbuilders together with owners' naval architects have worked out wonderful docking schemes, with a forest of blocks placed at the intersections of transverse and longitudinal strength members in the ship's bottom. Such schemes, however, prevent the use of any kind of mechanization for cleaning and painting, and what is worse, they prevent paint from being applied to certain areas of the bottom during at least every second docking. For example, one scheme developed for tankers of 300 000 tonnes dwt, involved over 1100 blocks in 13 rows, leaving about 1500 m^2 of the ship's bottom unpainted year after year.

Generally speaking, it should be possible to carry the total weight of any ship, with bunkers and enough ballast, on three rows of blocks placed at the centreline and under or close to the longitudinal bulkheads. The load should be distributed 50–60 per cent on the centreline blocks and 20–25 per cent on each row of side blocks. The ship can be placed with an accuracy of 100 mm in the fore-and-aft direction and about 50 mm in the athwartships direction by using simple mechanical devices.

Lately a number of very big floating docks have been constructed or are under construction and it is therefore extremely interesting to note the differences in the behaviour of a ship in such a dock, compared with a graving dock. It is quite obvious that a floating dock will be much more flexible than a graving dock, and will follow the ship's deflected form more faithfully. It should be possible to design a system whereby the floating dock, together with a limited number of keel and bilge blocks, can give much more satisfactory support to the ship than the very stiff graving dock, with its complicated system of blocks.

At Götaverken, we are currently very busy with the design of a completely new system for the support of ships in dock, in close collaboration with Lloyd's Register and the builder of our mammoth floating dock. We hope to be able to present the technical details of our solution within a year's time.

Work in drydock

The jobs involved in the cleaning and painting operations, as well as the inspection of tailshaft and rudder, tend to be similar for all ships and may be mechanized to a great extent. Cleaning systems have been improved by the introduction of new high-pressure water or brushing methods. Surface preparation is now carried out by sophisticated machines and painting is done from very efficient hydraulically-operated platforms (*see* Figs 1–4). We may confidently foresee further developments in collaboration between shiprepairers, owners, Lloyd's Register and paint manufacturers, leading to a reduction in the time required for the normal 'haircut and shave' and providing improved quality.

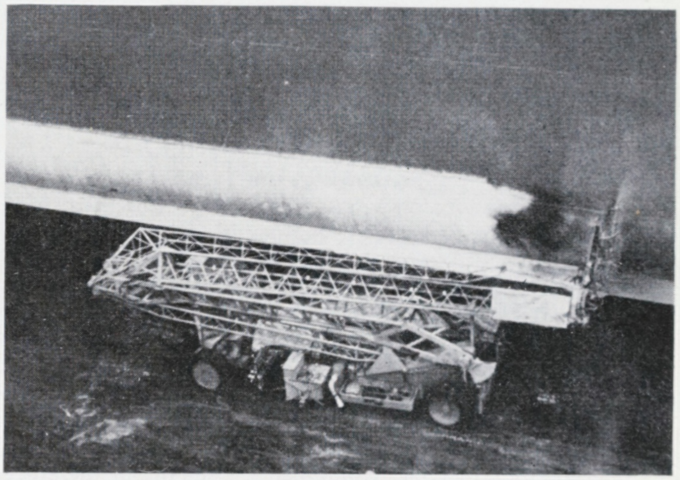

FIG. 1

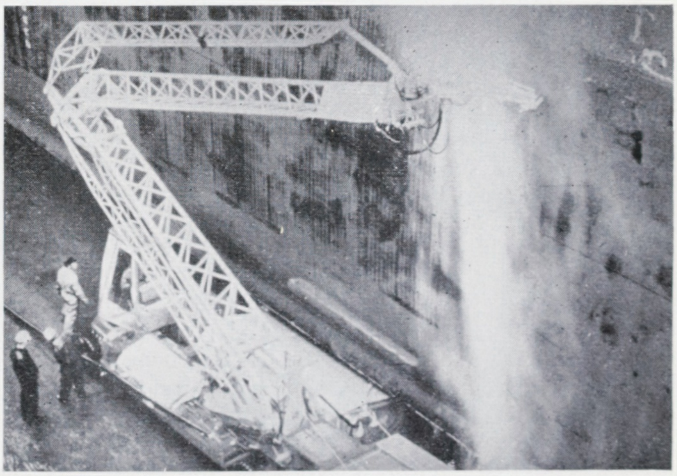

FIG. 2

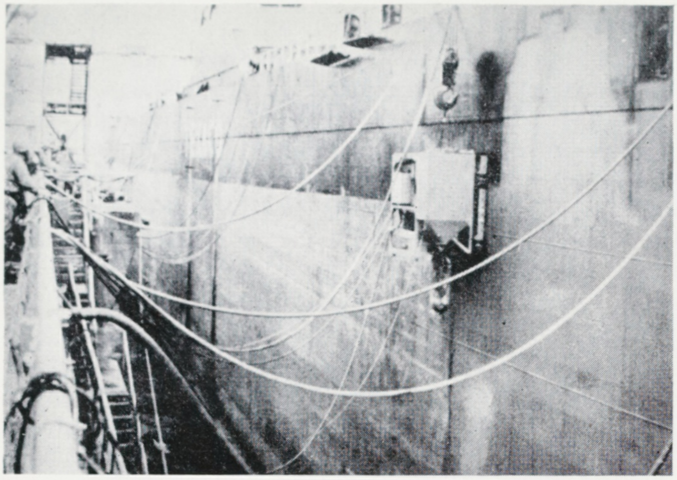

FIG. 3

Besides this routine work of maintaining the ship's bottom in smooth condition, drydocks are called on now and again for repairing damage due to heavy weather, groundings, collisions, etc. Shiprepairers tend increasingly to take a leaf from the newbuilders' book and utilize section building to a greater extent. To be able to use section building efficiently for the repair of bottom damage, the block height should be increased so that heavy sections can be introduced under the bottom of the ship at the point of damage. At the moment Götaverken are experimenting with the use of the air cushion principle to move damaged parts and new sections and hope in this way to be able to minimize the necessary time in drydock. Another method of effecting the repair of very severe bottom damage is to cut out the damaged part of the bottom and then undock the ship with tank hatches and openings closed, floating on the entrapped air. Subsequently, the ship can be redocked on top of a new bottom section, either in another dock or in the same dock after a certain waiting time alongside a berth, while the damaged bottom section is being scrapped and a new bottom placed on the blocks (Fig. 5).

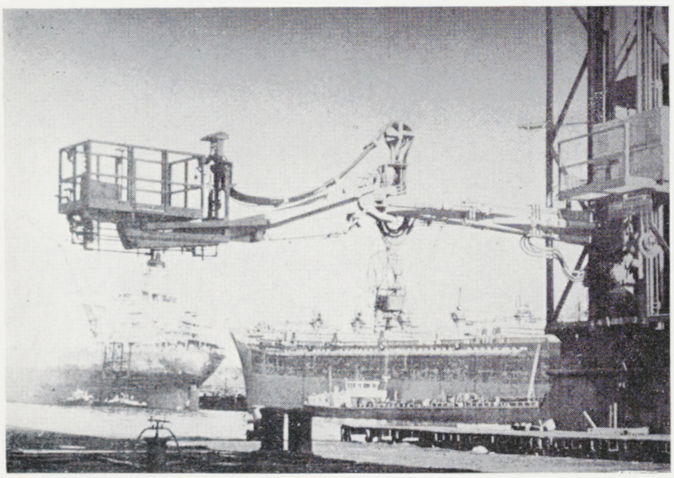

FIG. 4

These repairing methods imply that, if possible, a damaged ship should be temporarily repaired only sufficiently to enable her to continue trading for the time required by the shiprepairer to prepare the sections to be replaced. This will require close co-operation between owner, underwriters and Lloyd's Register, as well as the yard.

Shiprepairers' wishes for the future from Lloyd's Register

First of all, the necessity for repair work to be carried out round the clock is emphasized and hence the Lloyd's Surveyor should be prepared for a 24-hour day service to attend to ship repairing. Secondly, it is pointed out that modern ships have forced the repairer to employ a new type of manager with a more theoretical education than before and it is also hoped that Lloyd's Register will not choose Surveyors with practical experience alone. Complementary education after practical experience normally results in a first class Surveyor. Thirdly, it would be appreciated if the local Lloyd's office could be

FIG. 5

Replacement bottom sections on blocks.

supplied with necessary drawings and copies of internal damage reports of the ship to be repaired and that the local office could be permitted to discuss even the hitherto secret reports freely with the shiprepairer.

Examinations on board a ship undergoing repair sometimes reveal unexpected defects or wear down, the correction of which may require theoretical calculations. It is essential that back-feeding to the Surveyor acting on the spot is such that least possible delay is caused. In Gothenburg there is very good experience in this field but it may not be so everywhere.

Continuous surveys and on-voyage repairs

It is necessary to maintain the propulsion machinery and cargohandling equipment in good condition, not only in order to avoid unnecessary drydocking expenses, but also to eliminate unscheduled off-hire time. Continuous maintenance of such

equipment is therefore carried out by crews and checked by Lloyd's Register. In many cases the survey may be carried out not only during the ship's scheduled stops at loading or unloading ports, or repairing yards, but even on voyage. Tank testing and inspections may often be performed under better conditions while the ship is in operation. It has become more and more common for Surveyors to accompany ships on voyages. This practice, combined with 'flying repair squads' may make it possible in many cases to rectify even major damage to equipment and even cracks in internal steel in cargo tanks, without recourse to a repair yard. With the tendency to minimize ship running costs to that necessary for safe navigation, there will be an increased need for specialized repairers who can follow the ship to any part of the world.

Swedish repairers have therefore recently started building a worldwide chain of service stations capable of coping with any normal need. This chain is intended to be the basis for future development in ship repairing and maintenance. In the same way as a car manufacturer cannot build up his market without having the area covered with service stations and spare parts, it is the intention to build up a world-wide chain of service stations that can supply ships with harbour and voyage repairs between drydockings. At each of these stations there will be placed an expert on the builders ships, who can advise the ship's officers and, if required, supervise the repair work.

Repairs and maintenance in the future

Yet further developments are indicated by the introduction of a five-year maintenance contract scheme. The philosophy behind this is that, in order to be able to develop still better ships, it is necessary to ensure feedback not only during the guarantee years but during at least the first five years of a ship's life. Because the quality of the ships delivered will be known it will then be possible to estimate the normal repair and maintenance cost, as well as to make a better assessment of the relative capabilities of subcontractors.

It is, therefore, the intention to offer shipowners placing orders with Götaverken the opportunity of signing a five-year maintenance contract, which will include a follow-up of the ship's performance, all kinds of voyage repairs not normally done by the crew, and also drydocking, spare parts, etc. Drydocking will be offered not only at Gothenburg but also at at least three more points around the world, under agreements between Götaverken and other repair yards. Needless to say, this scheme means still closer collaboration with Lloyd's Register and there still remains very many problems to be overcome together with various owners, as well as underwriters, before such a maintenance scheme can become a reality.

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Lloyd's Register Technical Association

Discussion

on

Mr. Th. Andersson's Paper

LLOYD'S REGISTER AND THE SHIPREPAIRER

Paper No. 4. Session 1975-76

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Discussion on Mr. Th. Andersson's Paper

LLOYD'S REGISTER AND THE SHIPREPAIRER

Mr. J. McCallum

It is a great pleasure for me to see and welcome Thorsten Andersson, a much-travelled and highly respected member of the West European community of shipbuilders and shiprepairers. One of the things that impresses on reading the paper and listening to his presentation is his ability to put a mass of information into small bulk—a great virtue and no doubt the result of his shiprepairing experience.

Referring to a few matters arising in the paper I would comment as follows: —

Inspection of internals of cargo tanks from rubber dinghies is a very dangerous operation in my opinion, particularly in any sort of a seaway.

I can understand Mr. Andersson's suggestion regarding a standard rise of floor as a shiprepairer, and no doubt somebody else will take this up later this evening, but I am rather baffled by the argument of gradient in preference to rise of floor in the half-width, but if he likes it that way, that's fair enough!

I confess I wouldn't much like to see a Lloyd's Register classed ship on a bed of nails (as the Author refers to dry docking) unless they were pretty blunt, but I can see that in terms of a floating dock some transverse flexibility would be desirable. For those old-fashioned ships with rise of floor, is this amount of transverse bending—i.e. 2″—going to help greatly?

I think the load measuring capsules are a good idea but as Mr. Andersson says, the readings are very dependent on how flat the bottom is. I fancy he has his tongue in his cheek when he asks about the maximum variation in bottom waviness in a newbuilding. Well, of course, it's not just Lloyd's Surveyors who deviate from the average: shipbuilders tend to do it too. But in the maximum case, I would expect the worst deflection to be about the same as the plate thickness, but it must depend on how much the supporting member above has deflected Every case has to be assessed on its merits, and I imagine that Mr. Andersson's views now as a shiprepairer might be slightly more conservative than when he was a shipbuilder.

I must say, he raises visions of a most provocative nature and I shall make a point of letting it be known that I would like to witness the highly unorthodox 'mattress' trials on the new floating dock.

On another point raised, the Surveyors will turn out at all times on request; it's only a matter of having sufficient staff on call and having reasonable forewarning.

I believe all our appointments recently have been well-qualified naval architects and if some are a bit thin on the practical side, we can polish that up quickly.

We don't run a secret service, that is, not so you would notice, but a lot of our business is based on integrity, and integrity means that you don't wave plans around which are other people's property unless you've got their permission first: and that's all that's needed.

Mr D. Gray

I find it perfectly natural that the bulk of the paper and all the discussion so far should be concerned with steel-work problems. On the other hand an unbiased observer may be forgiven if he concluded that failures occurred only in the steel-work area and that the machinery was trouble free. This is entirely untrue of course.

The author had asked for comment and feedback of information from the audience. It is my opinion that the major problems in the ship repair industry, so far as electrical engineering and control engineering is concerned stem from the following factors: —

(i) Lack of proper facilities and test equipment for electrical engineering and control engineering work.

(ii) General cleanliness is not good enough for satisfactory work to be carried out in the electrical and control field.

(iii) The quality of labour in the electrical and control fields is frequently lower than in the new construction industry.

(iv) In many cases there appears to be an attitude of mind that electrical and control engineering work is of secondary importance compared with, say, hull and main machinery.

(v) Short turn round times.

(vi) In many cases the job appears to be costed at a minimum price.

It would be of interest to hear Mr. Andersson's views on these points.

Coming to specific problems, there are many items of electrical equipment and even more so in the control engineering field where individual units, fitted in the ship when built, are no longer available after some years. This, of course, may be due to advancing technology or to re-organization within a manufacturing area. Replacement with a modern equivalent can be very costly. In some cases it is not practicable due to different physical dimensions. How does the ship repairer cope with such a problem?

One final question for Mr. Andersson and it is a very general one. How does the repairer ensure continuity of work so that he can hold his labour force together?

Mr. W. H. Marsden

I congratulate the Author on the refreshing way he has shown how certain Shiprepairers have reacted to the rapid change in shipbuilding technology, which has occurred during the last decade.

It is unbelievable now, that the highest practicing technology in ship's strength ten years ago was devoted to the approximate formulation of Rule criteria based only on extrapolated experience. I believe, that the Society in association with Industry has the necessary financial and staff dedication which has enabled computer technology to be fully used in the assessment of ship structures. This technology changed the outlook and approach in both shipbuilding and shiprepairing and countries which were expanding appreciated the opportunity of this technical leap forward.

Sir, from your paper I can see the opportunity was taken by you, but with due respect there are others in your industry who still consider ship repairing is simply to prepare specifications with owners and refer it to the Surveyor when it is

decided. As you so correctly emphasise in your paper, co-operation and prior planning is the key to a succesful repair team; my experience, under the present market conditions, indicates that owners are also becoming more aware of the advantage of this early co-operation with classification societies.

I was particularly attracted to the paragraph in your paper on repairs and maintenance in the future.

During the last two years, one of the Society's working groups on the Survey of large tankers has been examining the prospect of planned maintenance in which the possibility of 'Monitoring Surveys' was considered. These are proposed surveys carried out at more frequent intervals with details fully recorded and transferred to computer for record. This information then enables the planning of repairs or renewals with the owners at the due date. The important problem to some was the arranging of staging. A proposal was made to have walking galleries, frequently spaced within the hull depth, so that Surveys can be held at sea during free time in ballast. It would be of interest to have the Author's views of the repairers approach to provision for staging in large tankers especially with respect to maintenance schemes and classification requirements.

The Author's proposals concerning the docking of a VLCC is fully concurred with. As we are aware, the position of docking blocks and distribution of ballast in the docking condition requires to be examined before the final approval of the scantlings of an intercostal docking girder on a tanker. It is obvious from certain docking conditions submitted that only trim and stability have been considered and not the resulting load distribution on the blocks and ship structure. There has already been a number of damages to bottom structure when the specified ballast distribution or block arrangements has not been used.

One of the problems facing an owner, is that he is rarely aware during the building stage, where his tanker will be docked during service. Therefore the block arrangement supplied by builders for classification purposes may in fact not be the arrangement used when the tanker first docks. This indicates how essential co-operation is between the owners, repairers and classification societies.

Finally, as the Author states, the docking operation is the nucleus of any ship repair; can the Author advise who is responsible for the docking condition of a tanker? My reason for putting this apparently naive question, is that recently, following a damage to the continuous centre girder and transverses of a three girder centre tank system, it was reported by the shiprepairer that the owner was responsible for the ballast condition and the repairer for the block arrangement. Theoretically, this is an impossible situation for a satisfactory ship structural solution.

Mr. R. F. Munro

I can find only two references to machinery items in the paper and these are confined to screwshafts and sea connections. It is understandable that the Author has placed so much emphasis on the problems of drydocking very large vessels because this is a subject with which he has much experience, however the repair of machinery and boilers is also a very important part of the shiprepairers' activity requiring a well organized department employing specialists in a number of engineering fields if ships are to be ready in all respects to leave the repair yard on schedule. The Surveyors have an important part to play here and there is no doubt that much is to be gained by meetings between owners' superintendent, repair manager and surveyor prior to the ship's arrival at the repair yard, and repeated at intervals as the repairs progress. In this way the possibility of items being opened up and overhauled but not credited in the classification survey records is greatly reduced. While there are a number of large ships in which surveys of screwshafts can be held afloat, and while some owners are developing means of opening up sea connections for examination during in-water surveys, the vast majority of these surveys will undoubtedly be dealt with in the traditional manner in dry dock far into the future.

Due to the increased interval between dockings now permitted by the Rules it is very important that the due dates of survey of screwshafts and sea connections should be considered when a ship is placed in drydock. Sea connections, as part of the Continuous Survey of Machinery (CSM) fall due for survey every five years and cases of acute embarrassment to all concerned have arisen when ships have undocked with sea connections and screwshafts due, or very nearly due for survey. In order to minimise the incidence of such oversights in future the Society has arranged for the due dates of sea connection surveys to appear on all computer output of CSM cases in addition to all the main survey dates, including screwshafts, which already appear so that owners and surveyors alike are fully aware of the situation.

It may be stating the obvious that when ships are in drydock the weardown of the screwshaft or clearance in sternbush, and/or efficiency of the oil gland where applicable, should be recorded without fail. It is the duty of the Surveyor to ensure that this is done and to forward the results in his report to the Committee. The Author is thanked for giving this further opportunity of drawing attention to the importance of this matter. It should be borne in mind that the complete history of a screwshaft must be known if a proper technical assessment is to be made of the merits of an application for postponement of the survey. At routine dockings which are planned in advance these matters are normally allowed for; it is at forced drydockings for damage repairs that they are more likely to be overlooked.

While it is agreed that the classification society is the only impartial body in the four-handed game at the repair yard and is free from direct commercial interests it must not be concluded from this that the Surveyors are not concerned with seeking the least expensive solution to repair problems consistent with acceptability. In these difficult times every member of the staff is repeatedly reminded of the obviously serious problems faced by ship operators.

In recent years the effort devoted to planning machinery repairs must have been reduced by the production of standard repair production charts by the major diesel engine designers. These show clearly in diagrammatic form the various stages in dismantling and replacing engine components and include tests of all the tools and lifting gear required together with the number of men and total hours. Much of the advantage to be derived from this procedure will be lost if the Surveyor is not told in advance when he is likely to be required on board or in the workshop to play his part, in order that he may plan his day to suit, so far as possible, all the clients requiring his services.

The Society has been quick to react to the needs of owners to have repairs carried out at sea and the Surveyors throughout the world have received guidance in order that such work

should be started, carried out, and finally checked in a uniform and acceptable manner for maintenance of the ships' class. Furthermore, the Society's Surveyors are being called upon with increasing frequency to travel on board ships for the purpose of carrying out periodical surveys of many kinds. Whereas twenty years ago the office car was becoming the object of some discussion as to who was responsible for maintaining the oil level in the sump, our outport colleague of today in certain areas is perhaps more likely to be making radio contact with the nearest helicopter service.

WRITTEN CONTRIBUTIONS

Mr. R. G. LOCKHART

Mr. Chairman.

I apologize to you and my friend Thorsten Andersson for my unavoidable absence tonight.

The Author is a man of few words and much action and seldom if ever has so much been said to the Technical Association in so few words. I will try to be equally brief in my comments which are: —

1. Those responsible for dry docking ships do not always pay enough attention to the particular structural arrangement of the ship under consideration.
2. The effect of ballast on the structure in relation to the blocking arrangement is not always sufficiently considered. We have seen some examples of the damages accruing.
3. I would suggest that in the case of a VLCC with deep transverses in the centre tank, the weight distribution on three lines of blocks would likely be about 40 per cent on the keel blocks and 30 per cent on each line of blocks under the longitudinal bulkheads.
4. With insufficient blocking under the longitudinal bulkhead buckling of the bulkhead could occur due to weight concentration.
5. Can Mr. Andersson expand on the experiments with the air cushion principle?
6. The method of fitting bottom structure in Fig. 5 should be economic and minimise time spent in dry dock. Is the strength of the ship with the damaged bottom structure removed and floating in the entrapped air fully investigated? It would appear that high stress concentrations in way of bottom openings could be a feature of this principle.

My thanks to the Author, who is one of the world's authorities on dry docking of ships, for his enlightening and comprehensive paper.

Mr. F. N. BOYLAN

To the welcome of our President I would like to add my own, and thank Mr. Andersson for preparing such an interesting paper and coming all this way to read it to our Association. It is very helpful to listen to people from outside Lloyd's Register when they discuss the work of the Society. We have had other examples of this in recent years and it helps us to see ourselves as others see us.

Mr. Andersson, in a fairly short paper, has had to express himself briefly, and in some instances has presented perhaps an over-simplified view of the work done by the Society's Surveyors. The decision of a classification society is described as 'law' and, whilst I accept that the Author is knowingly using the word loosely here, it cannot be over-emphasized that a Surveyor does not have the authority to order work of any kind to be carried out. He can recommend whatever he considers to be correct in the circumstances but he cannot command anything. Lloyd's Register takes great care to remind its Surveyors that they are dealing with the owner's property, and as Mr. Andersson points out, it is always possible to discuss a Surveyor's recommendation and come to an acceptable and amicable agreement whenever a difference of opinion occurs.

On the matter of drydocking, it used to be customary to drydock a ship every year, and many companies operating passenger or cargo liner services for which it was essential to maintain uniform service speeds used to drydock more frequently than this, some even every six months. First, with improvements in anti-corrosive and anti-fouling paints, and later with the introduction of impressed current cathodic protection systems, the necessity for such short docking intervals disappeared, and it is now customary to drydock at about two-yearly intervals. However, Special Surveys are generally being carried out at five-yearly intervals and, as drydocking is a necessity for such surveys, docking at two and a half year intervals, intermediate between Special Survey dates, is much to be desired. This is tending to become the normal interval between drydockings. The requirements of the National Authority under which each ship is registered must, of course, be complied with, and there is as yet no official acceptance of two and a half year docking intervals. The Society has, however, a clause in its Rules to the effect that where a suitable high resistance paint is applied to the underwater portion of the hull and an approved automatic system of impressed current cathodic protection is fitted, the maximum interval between drydockings may be extended to about two and a half years at the discretion of the Committee. Usually, any well-kept ship can obtain this extension, and once this situation is officially accepted there should not be any valid reason for extending the drydocking intervals because to do so would bring dockings out of step with Special Surveys.

Where VLCC's are concerned we have, of course, helped to develop the practice of in-water surveying in lieu of drydocking, but this has very real limitations. Firstly, in-water surveys can be effected only where the water is adequately clear, and the prevailing weather conditions provide prolonged periods of sunlight. Various locations on the normal routes of very large tankers between their loading and discharging points have been developed for this purpose. The greatest limitation, however, is that it is not yet practicable to re-coat large areas of plating under water, and until this can be achieved, a vessel which has been thoroughly cleaned below water will foul up again almost immediately so that loss of speed is felt after only one trip. For this reason also, in-water surveys have been limited in practice to the survey intermediate between delivery and the first Special Survey. There is no doubt that the supporting of very large ships in drydock poses some very serious problems, and I am intrigued by Mr. Andersson's comments on the new system which Gotaverken has developed. It would be nice to think that in a year or two perhaps the Author will address us again and give us the benefit of his experience with the new system.

Turning now to Mr. Andersson's comments on the service that shiprepairers would wish from Lloyd's Register in the

future, I must first of all say that Lloyd's Surveyors, particularly those engaged on repair work, have always realized that they are liable to be called upon at any time around the clock. This they more or less cheerfully accept, provided, of course, that when possible, reasonable notice is given of attendance outside normal business hours, and that attendance without prior notice will be asked for only in cases of real emergency. I am sure Mr. Andersson will realize that a Surveyor is at the call of many clients, and if all of them regarded him as being available without notice at any time he would have a somewhat thin home life.

Regarding the qualifications required to be possessed by candidates for posts as Surveyors to Lloyd's Register, I can tell Mr. Andersson that it is a long, long time, if indeed it was ever the case, since the Society recruited its Surveyors on the basis of practical experience only. Some practical experience is still desirable, but the modern Surveyor is required to have a wide range and a high standard of technical preparation. Further training in both practical and theoretical work is provided within the Society by giving Surveyors experience in outports under the guidance of senior colleagues, and also by posting to the various technical sections in London or other national Headquarters. Lloyd's Register also operates two specially organized Training Centres, one in the United Kingdom and one in Japan, where a constant flow of Surveyors from all over the world attend a variety of courses to familiarize them with the latest developments in shipbuilding and engineering techniques.

On the question of the supply of drawings and copies of damage reports to Outports it must be remembered that there are over 100 million tons of shipping classed with the Society. This respresents many thousands of ships, and I am sure Mr. Andersson will realize that to send copies of drawings and reports, as a matter of course, to ports all over the world every time a ship goes into drydock would be an almost impossible undertaking, and that even were it possible, the expense would be enormous. I may say, however, that in connection with major damages or cases of particular difficulty, a Surveyor can always obtain whatever information he requires from Head Office with the minimum of delay by sending a request by cable or telex. I think when he refers to 'secret' reports, Mr. Andersson really means confidential. All reports in connection with particular ships are confidential documents and must not be disclosed to third parties without the permission of the owners. If an owner has no objection, however, there is no reason why such reports cannot be discussed with the repairer working on his ship.

The practice of carrying out surveys at sea, during the ballast voyages of tankers for example, is one which some owners have found to be worthwhile, but there are obvious limitations. Such surveys cannot be carried out in weather which produces much movement of the ship, and a great difficulty is in examining structure at any considerable height above the bottom of tanks.

With regard to the last section of his paper, Mr. Andersson might be interested to know that Lloyd's Register has instituted a feed-back service very much on the lines visualized by him. We have been in touch with several owners, and they will arrange to monitor the wastage-rate due to corrosion with exactly the intention he indicates. The Society also has a system by which particularly noteworthy defects are recorded in our Technical Records Office, and a monthly booklet covering these is issued for the information of Surveyors. It is hoped by this means to overcome the problem of the repetition of undesirable structural details.

AUTHOR'S REPLY

I would like to answer some of the very interesting and intriguing questions asked by the members of Lloyd's Register Technical Association.

To Mr. McCallum.

I should like to point out that according to our theoretical calculations the possibility of bending the dock transversely by ballasting centre tanks and emptying side tanks will be something like 55 mm over the half-width of the dock.

To Mr. Gray

I certainly agree with Mr. Gray that shiprepairing is not only concerned with steel-work but also with machinery and lately an increasing amount of electrical and control engineering work. I would like to reiterate the necessity for cleanliness and concur that electrical and control engineering are becoming more and more important. As to his final point about ensuring continuity of work, this is, of course, most critical in ship repairing. We are trying to overcome these difficulties by making certain sections of our new building yards more fluid, in order to try to secure certain basic jobs, such as conversions. Furthermore, a yard with a large turnover is, in my experience, easier to keep in balance than smaller ones.

To Mr. Marsden

Mr. Marsden raised a very intriguing question about who is responsible for the docking conditions of ships. In our opinion it is the responsibility of the shipyard to arrange sufficient numbers of blocks to support the vessel. Here we are guided by Lloyd's Register's recommendations of average block pressure. It is the owners' responsibility to advise the shipyard of the structure of the ship, its proposed ballast conditions, placing of bunkers etc. Futhermore, it is the owners' responsibility to ballast the ship so that it can be safely handled during manoeuvres and to de-ballast the ship during the docking operation in order to avoid unnecessary stresses on the ship.

To Mr. Lockhart

Mr. R. G. Lockhart made a number of observations for which I am very grateful. At present we are working on the problem of keel block arrangements and also on keel block

design, as apparently the caping which we used at our first docking was too soft. It is necessary to stress the point that even if the ship has to be ballasted in a certain condition to have the right draft and trim for safe manoeuvring and to minimize stresses on the hull, it must be clearly understood that this ballast should normally be taken out of the ship during the docking operation after safely setting on the blocks. With regard to the experiments with the air cushion principle, until we have had some practical experience, I consider it premature to comment at this time.

To Mr. Boylan

I would like to thank Mr. F. N. Boylan for his observations regarding the limitations of Lloyd's Surveyors and his clarification on the limits of in-water surveys and underwater cleaning.

Finally, I would like to thank all those who participated in the discussion and especially Mr. McClimont for giving me this opportunity to present the viewpoint of the shiprepairer before your Technical Association.

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Lloyd's Register Technical Association

CONCRETE CONSTRUCTION

D. L. Schonhut

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Hon. Sec. A. Wardle
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CONTENTS

CONCRETE CONSTRUCTION

by D. L. SCHONHUT*

1 INTRODUCTION

Lloyd's Register of Shipping became involved in the quality assurance of concrete structures during the First World War. This was for the classification of several types of concrete ships and barges. Since that time the involvement in concrete construction has increased and included floating docks, offshore concrete gravity platforms, pontoons, anchors, etc.

This paper is for the guidance of those concerned with the construction and supervision of concrete work. It is not intended to replace specifications, Codes of Practice, Bye-laws or other regulations.

2 MATERIALS

Concrete is made from cement, aggregate and water with the occasional addition of an admixture. There is some variety in the properties of cements, even between cements of the same type but made from different raw materials, and the variation in properties possible in natural and manufactured aggregates is almost limitless. Concrete is always a heterogeneous material with variable properties.

2.1 Cements

The most widely used cement in the world is ordinary Portland cement and it accounts for about 90 per cent of all cement production. It is made by heating limestone and clay, or other suitable raw materials, together to form a clinker rich in calcium silicates. This clinker is ground to a fine powder with a small proportion of gypsum (calcium sulphate) which regulates the rate of setting when the cement is mixed with water. Several varieties of Portland cement have been developed: rapid-hardening, sulphate resisting, white and low heat. They all contain the same active materials—only the proportion of each is different and they exhibit special characteristics or properties which are of value in appropriate circumstances.

By incorporating other materials during manufacture an even wider range of cement is produced: Portland-blastfurnace, masonry, coloured, oil-well, water-repellent and hydrophobic cements.

The setting of cement is a chemical reaction between cement and water, not a drying process. This reaction is called hydration. It evolves heat and is irreversible. Setting is a gradual stiffening process which is defined by arbitrary limits. Strength continues to gain after hardening and may take many years to reach its ultimate value.

2.1.1 Portland cements

2.1.1.1 Ordinary and rapid-hardening Portland cement

Rapid-hardening Portland cement (RHPC) is ground somewhat finer than Portland cement (OPC). The finer cement gains strength more quickly but after several months there is little to choose between the two.

2.1.1.2 Sulphate-resisting Portland cement

Sulphate-resisting Portland cement (SRPC) is a form of Portland cement with a low tricalcium aluminate (C_3A) content. It usually has a higher content of tetracalcium aluminoferrite (C_4AF) than other Portland cements, which gives it a darker colour. Because sulphates can react with hydrated tricalcium aluminate, causing weakening, concrete made with this cement is more resistant to attack by the sulphate compounds which may be found dissolved in ground water and which are present in sea water.

* Structural Engineer, Headquarters.

TABLE 1

REQUIREMENTS FOR PORTLAND CEMENT CONCRETE EXPOSED TO SULPHATE ATTACK

Class	Concentration of sulphates expressed as SO_3: In soil: Total SO_3	In soil: SO_3 in 2:1 water:soil extract	In ground water	Type of cement	Minimum cement content, nominal maximum size of aggregate: 40 mm	20 mm	10 mm	Maximum free water/cement ratio
	%	g/litre	parts per 100 000		kg/m³	kg/m³	kg/m³	
1	less than 0·2	—	less than 30	Ordinary Portland or Portland-blastfurnace	240	280	330	0·55
2	0·2–0·5	—	30–120	Ordinary Portland or Portland-blastfurnace	290	330	380	0·50
				Sulphate-resisting Portland	240	280	330	0·55
3	0·5–1·0	1·9–3·1	120–250	Sulphate-resisting Portland	290	330	380	0·50
4	1·0–2·0	3·1–5·6	250–500	Sulphate-resisting Portland	330	370	420	0·45
5	over 2	over 5·6	over 500	As for Class 4, but with the addition of adequate protective coatings of inert material such as asphalt or bituminous emulsions reinforced with fibreglass membranes.				

Requirements for dense and fully compacted concrete

Concrete made with SRPC has been found to be satisfactory in nearly all troublesome conditions which arise with below-ground concreting. However, resistance to sulphate attack depends on the cement content and imperviousness of the concrete as well as on the concentration of sulphate encountered.

A summary of recommendations is shown in Table 1.

SRPC is similar to other Portland cements in that it is not resistant to acids, nor is it immune to the effects of some other dissolved salts, such as magnesium compounds, which may occur in natural waters or effluents.

2.1.1.3 **White Portland cement**

White cement is distinguished by its low content of iron compounds which impart the grey-green colour to ordinary cements. It is made by using white china clay and limestone as the raw materials. Gypsum is added to control setting, and special care is taken at all stages of processing not to introduce coloured contaminants. It is used for decorative white concrete and also for some coloured concretes, in which case a pigment is added to the mix.

2.1.1.4 **Low heat Portland cement**

Low heat Portland cement (LHPC) produces concrete which gains strength and evolves heat more slowly than normal concrete of similar composition, though ultimately the strength and heat of hydration are virtually the same. It is intended for use in large masses where the rapid evolution of heat would cause high temperatures and stresses which might lead to cracking. The ratio of compounds present is different from other cements and its special qualities call for a separate specification.

LHPC is not normally available except to special order and it is usually considered only for very large structures. For smaller works where the effects of excessive heat of hydration may be a problem it is worth considering the use of sulphate-resisting Portland cement as this usually has a lower rate of heat evolution than ordinary Portland cement. As mentioned above, the total heat evolved by most Portland cements is similar, but the rate at which it is evolved is the important parameter to consider when heat of hydration poses a problem in construction.

2.1.2 **Cements based on Portland cement clinker**

To meet further special demands, cements are marketed which are based on Portland cement clinker. The name Portland is often not retained in the title because the cements contain chemical additives which are, by definition, excluded from Portland cement.

2.1.2.1 **Masonry cement**

Mortars, if made with ordinary Portland cement and sand, tend to be too harsh or too strong for rendering and brick– or blocklaying. It has been customary to overcome this problem by adding lime to the mix, but nowadays masonry cement is available. This consists of ordinary Portland cement with the addition of a fine powder and a plasticizing agent. Masonry cement is not a suitable substitute for ordinary Portland cement and lime for mortars of very high strength, and should never be used in concrete.

2.1.2.2 **Hydrophobic cement**

In this cement (available only to special order) the particles are coated during manufacture with a waterproof skin of oleic acid or other water-repellent to resist hydration in case the cement has to be stored for a long time in a moist atmosphere. This coating is rubbed off by friction in the mixer, and the cement then behaves normally. In most cases, less than 0·5 per cent of additive is required to obtain the designed effect. The added material may also serve as a plasticizer in the concrete mix.

2.1.2.3 **Portland-blastfurnace cement**

This cement is a blend of Portland cement and suitable granulated blastfurnace slag in proportions of 2 to 1 or thereabouts.

The mechanical properties of the cement are similar to those of ordinary Portland cement, but concrete made with it tends to gain strength more slowly and less heat is evolved in the process; it is often claimed to be more resistant to chemical attack. A special grade, low heat Portland-blastfurnace cement is intended as an alternative to normal low heat Portland cement.

2.1.2.4 **Water-repellent cement**

If a metallic soap is mixed with cement, concrete made with the cement becomes water-repellent and tends to shed the dirt in rainwater better than normal concrete. This is particularly important with decorative concrete and cast stone which has an open pore structure. Both white and grey water-repellent Portland cements are available and are used in precast concrete as well as in other branches of the construction industry. Water-repellent and hydrophobic properties can be combined in the same cement.

2.1.2.5 **Oil-well cement**

Until recently, oil-well cement was made primarily for export, but the discovery of oil and gas in the North Sea has brought its use nearer home. Several grades are marketed, designed specifically for cementing in steel casings or for plugging shallow, medium and deep boreholes in which high temperatures and high pressures are present. Apart from these specialized applications, oil well cement is not used in the construction industry.

2.1.3 **Delivery and storage of cements**

Cement may be supplied in bulk, in bags or in drums. Bulk cement is delivered by tanker, usually in loads of 10 tonnes, and blown into storage silos on site by compressed air. Bagged cement is commonly supplied in bags containing 50 kg (110 lb), though special cements may be supplied in bags containing other quantities. Reference may sometimes be seen to the American sack as a unit, and the cement content of American concrete is frequently expressed in sacks per cubic yard. The U.S. sack contains 94 lb (42·6 kg). It is often convenient to use bags on the smaller site, but cement is cheaper in bulk. Cement is normally supplied in drums only when long storage is expected or it is to be exported.

It is self-evident that cement should be kept dry during storage. Hydrophobic cement has only a small increased tolerance for damp compared with ordinary Portland cement. Problems of long-term storage are usually avoided by planning cement deliveries so as to anticipate only the short-term requirements. Storage in moist air leads to the phenomenon of 'air-setting', which results in the formation of lumps of hydrated cement. These lumps should be screened out and discarded if found in cement. Silos are normally weatherproof, but during prolonged periods of storage some air setting may occur due to condensation in the silo. This is minimized if the cement is circulated by withdrawing it from the bottom and pumping it back into the top of the silo; this should be

done frequently in periods of prevailing damp weather. In addition, the weather-proofness of the silo should be checked if there is any evidence of an increase in the lumpiness of the cement.

Bagged cement should be stored on a raised floor in a damp-proof shed in order to prevent deterioration. Failing this, it should be stacked on a raised timber platform and covered by waterproof covers with generous overlaps. The bags should be used in the order in which they are received; thus each delivery should be kept separate to avoid confusion. To avoid 'warehouse set', which results from the compaction of the cement, bags should not be stacked higher than about $1\frac{1}{2}$ m (5 ft). Although the paper bags used for packing cement are fairly waterproof, they are not vapour-proof, so undue exposure should be avoided. To avoid risk of accidental confusion, cements of different types should be stored separately.

Cement manufacturers will usually provide a certificate giving details of the properties of a delivery. Certificates should be filed for reference because the concrete mix design may need to be modified to take account of minor variations in properties.

2.1.4 **Sampling and testing of cements**

The testing of cement usually requires the resources of a well-equipped laboratory and should be carried out on site only in exceptional circumstances. However, if tests are required, cement will be sampled on site for submission to the test house.

A sample of cement taken for testing must be representative of the consignment and be taken within a week of delivery. It should be a mixture of at least twelve equal sub-samples taken from evenly spaced places throughout the consignment. For cement in bags or other packages, it should be a mixture of equal quantities taken from at least twelve bags, or from each bag when there are fewer than twelve bags. Sub-samples of bulk cement should be taken from the bulk container, or containers, during filling or emptying. The sample should weight at least 7 kg (15 lb) and be sealed in a clean airtight container. The relevant particulars should be marked clearly on the outside and the container sent to a suitably equipped laboratory. Testing should be completed within four weeks of delivery of the cement to the site.

2.2 **Aggregates**

The term 'aggregates' is used to describe the gravels, crushed stones and other materials which are mixed with cement and water to make concrete. As aggregates form the bulk of the volume of concrete, the selection of suitable material is important.

Gravels, sands and crushed stone, such as granite, basalt and the harder types of limestone and sandstone, are in common use as aggregates.

2.2.1 **Characteristics of aggregates**

2.2.1.1 **Essential requirements**

The two essential requirements of an aggregate are durability and cleanness.

Durability. Aggregates should be hard and should not contain materials which are likely to decompose or change in volume when exposed to the weather, or to affect the reinforcement. Examples of undesirable materials are coal, pyrites and lumps of clay: coal may swell, pyrites may decompose, causing iron oxide stains to appear on the concrete surface, and lumps of clay may soften and form weak pockets. High-strength mixes may call for additional special properties. In particular, the crushing value or impact value, density, or mineralogical type may be specified.

Cleanness. Aggregates should be clean and free from organic impurities: aggregate containing organic material makes poor concrete. The particles should be free from coatings of dust or clay, as these prevent the proper bonding of the material. An excessive amount of fine dust or stone 'flour' may prevent the particles of stone from being properly coated with cement and thus lower the strength of the concrete. Gravels and sands are usually washed by the suppliers to remove clay, silt and other impurities which, if present in excessive amounts, result in a poor-quality concrete. However, washing must not be carried to such an extent that all fine material passing the 300 μm (No. 52) sieve is removed, otherwise the resulting concrete mix will be lacking in cohesion and in particular may be unsuitable for a mix which is to be placed by pump.

2.2.1.2 **Types of aggregate**

The term 'fine aggregate' is used to describe natural sand, crushed rock, crushed gravel or other material, most of which passes through a 5 mm ($\frac{3}{16}$ in) sieve. 'Coarse aggregate' is the term used to describe material such as natural gravel, crushed gravel or crushed rock, most of which is retained on this sieve.

2.2.1.3 **Sizes of aggregate**

The maximum size of aggregate is governed by the type of work to be done. For reinforced concrete it should be such that the concrete can be placed without difficulty, surrounding all reinforcement thoroughly and filling the corners of the formwork. Hence it is usual for coarse aggregate for reinforced concrete to have a nominal maximum size of 20 mm ($\frac{3}{4}$ in). Where practicable, however, there are advantages in using larger aggregates. The size of aggregate may also be increased for foundations and mass concrete work. Smaller aggregate may be needed in concrete to be placed through congested reinforcement or similar restrictions.

In massive structures such as dams, use has sometimes been made of individual larger pieces of aggregate known as 'plums'. These are not mixed with the concrete, but placed in layers during pouring. This was a very labour-intensive operation and, because of this and other technical difficulties, plums are rarely, if ever, used today.

2.2.1.4 **Grading of aggregates**

The proportions of the different sizes of particles making up the aggregate are found by sieving and are known as the 'grading' of the aggregate: the grading is usually given in terms of the percentage by weight passing the various sieves. Continuously graded aggregates for concrete should contain particles ranging in size from the largest to the smallest; in gap-graded aggregates some of the intermediate sizes are left out. Gap-grading is not normally desirable for routine concreting work, though it may be necessary in order to achieve certain surface finishes.

For important work, the coarse aggregate should be obtained in separate single sizes rather than as one graded material, unless it has been reconstituted from separate sizes, accurately proportioned and pre-mixed at the source of supply. Where colour is important, supplies of aggregate should be obtained from one source throughout the job whenever practicable. This is particularly important for the fine aggregate, or for the coarse aggregate if an exposed-aggregate finish is required.

2.2.1.5 **Marine aggregates**

Some aggregates are obtained by dredging marine deposits. Most of these aggregates contain some shells and salt. These are not normally harmful in reinforced concrete, though limits may be set on the content of either. Marine aggregates should not be used for prestressed concrete.

Broken shells tend to affect the workability of the concrete, but are not themselves harmful. Limestone consists of compacted shells, and many limestones make very satisfactory aggregates.

A disadvantage of dredged fine aggregate is that, in many cases, it has a preponderance of one size of particle, which can make mix design difficult. Beach sands are often single-sized and can also have much higher concentrations of salt than dredged material because of the accumulation of salt crystals above the high tide mark. If beach sands have to be used, they should be washed carefully and their salt content checked frequently in a suitably equipped laboratory.

2.2.2 **Lightweight and manufactured aggregates**

In addition to natural gravels and crushed rocks, a number of manufactured aggregates are available for use in concrete. Air-cooled blastfurnace slag, which would otherwise be a waste product, is used in areas within haulage range of suitable steelworks.

Lightweight aggregates have been used in concrete for many years. The Romans made use of pumice in some of their construction work. Pumice is still used today and small quantities are imported and used in Britain, but most lightweight aggregate concrete is made using manufactured aggregates. All lightweight materials are relatively weak because of the porosity which gives them reduced weight. This imposes a limitation on strength, though this it not often a serious problem because the strength that may be obtained is comfortably in excess of most structural requirements. Lightweight aggregates are used to reduce weight in structural elements or to give improved thermal insulation.

In addition to lightweight materials, other manufactured aggregates have been used in concrete. These are mainly for special purposes; for instance, steel punchings or lead shot have been used to produce heavy concrete for radiation shielding.

2.2.3 **Storage of aggregates**

Aggregate should be stored so that it is self-draining and so that it does not become contaminated with other materials. If a clean hard base for the stockpile is not immediately available, then a layer of lean concrete should be placed, laid to falls in order to take water away from the stockpiles and also away from the mixer. Good drainage is particularly important for sea-dredged materials.

Stockpiles should be as large as possible as this helps to ensure uniformity of moisture content. Where possible deliveries should be allowed to stand in the stockpile for 12 hours before use. The use of aggregates from the lower part of the stockpile should be avoided since dirt and water from the higher layers can accumulate there. The various sizes should be separated from each other by dividing walls of sleepers or concrete blocks, and the practice of mixing alternate lorry-loads of, for instance, 10 mm and 20 mm material in the same bay should be avoided. It is frequently convenient to arrange the stockpiles radially with the mixer at the centre, although this does impose a limitation on the number of aggregates which can be stored.

2.2.4 **Sampling and testing of aggregates**

Testing of aggregates may be necessary, firstly to ascertain that the material is suitable for the required purpose before the job starts, secondly to verify that it is continuing to fulfil its requirements during the work, and thirdly to determine the water content so that the correct proportions of concreting materials may be calculated.

2.3 **Water**

Mixing water for concrete is usually required to be fit for drinking, or to be taken from an approved source. This is to ensure that the water is reasonably free from such impurities as suspended solids, organic matter and dissolved salts, which are frequently contained in natural water and which may adversely affect the properties of the concrete, especially the setting and hardening.

If there is any doubt about the quality of the water to be used for mixing concrete, it can be assessed by comparing, under similar conditions, the setting time of cement paste and the compressive strength of concrete made with the water in question and made with distilled water.

The use of sea water does not normally adversely affect the strength or durability of plain Portland cement concrete, but it is not recommended for reinforced or prestressed concrete because of the danger of corrosion of the steel, nor for concrete where efflorescence could mar the appearance of the work.

2.4 **Admixtures**

Admixtures are materials which are added to concrete during mixing. Their intention is to improve some property or properties of the concrete. The word 'additive' is normally reserved for chemical additions made to cement during manufacture and it is, strictly speaking, not interchangeable with the word 'admixture'.

Both admixtures and additives (using the above distinction) can confer benefits on a concrete. Indeed all Portland cements contain at least one interground additive, gypsum, without which it would be very difficult to control the stiffening of a mix within a reasonable period of time. The low cost of many admixtures is offset to some extent by the problems associated with handling and dispensing the admixture accurately at the concrete mixer. Overdosing, in particular, is to be avoided because excessive doses readily have adverse effects on the properties of the concrete. In some cases the benefits to be obtained by using an admixture could more easily be obtained by adjusting the proportions of other constituents of the mix.

There is a bewildering assortment of proprietary admixtures on the market under a host of brand names, but the active materials in them are relatively few in number. For convenience, admixtures may be classified according to their main purpose. In each class there is generally just one widely used active material.

2.4.1 **Accelerators**

Calcium chloride* is the most commonly used accelerator. It is used either on its own or contained in a proprietary admixture.

A serious drawback with calcium chloride is that it may lead to corrosion of embedded steel, which includes reinforcement and prestressing wires. Calcium chloride should never be used in prestressed concrete, and should not normally be used in reinforced concrete. Nor should it be used with sulphate-resisting Portland cement because the latter's sulphate-resisting

* BSI propose banning the use of calcium chloride in structural concrete.

properties are thereby impaired, a factor which is of importance when, for example, calcium chloride is considered for use in the mass concrete of a foundation.

Accelerators are ineffective in mortars because the quantity of cement in a mortar joint is too small to protect it from frost by accelerated heat evolution. In addition, calcium chloride can cause efflorescence which is particularly unsightly on brickwork. It may also lead to dampness in walls and to the accelerated corrosion of wall ties.

Accelerators are sometimes marketed under other names such as hardners, anti-freezes, frostproofers and even waterproofers. Altogether there are well over a score of brands of accelerator an the British market.

Calcium chloride itself may be purchased in solid form, either as the anhydrous salt or as 'flake', which contains water of crystallization. It should always be dissolved in water before being added to the concrete.

In order to avoid some of the disadvantages associated with calcium chloride some manufacturers are producing chloride-free admixtures.

2.4.2 **Water-reducing admixtures**

Some surface-active chemicals have the property of inducing a repelling force between cement particles and therefore act as dispersing agents in concrete. The dispersed particles require less water to lubricate them and the net result is that, for a given workability, a lower water content is required in the presence of these chemicals, which are known as water-reducing admixtures. In trade literature the terms densifiers, hardeners, waterproofers and plasticizers are sometimes used for these admixtures, on the grounds that the reduction in water content can lead to improvement in a number of properties of the concrete.

The most frequently used raw material for formulating this class of admixture is calcium (or sometimes sodium) lignosulphonate, a by-product of the wood processing industry. The material is also called 'sulphite lye' or simply 'lignin' in the trade; it is usually sold as a brown liquid with a distinctive and rather unpleasant odour. An addition of 0·20 per cent of the admixture by weight of cement will, in a typical case, enable the water content of a mix to be reduced by 10 per cent without loss of workability. This in turn allows the cement content of a mix to be reduced without loss of ultimate strength, which may lead to an overall saving in cost. However, to ensure sufficient durability, a minimum cement content may be specified, and care should be taken to ensure that the cement content is not reduced below this level.

The wood sugars present in lignosulphonate can be removed by processing, or they may be exploited to form a range of retarding grades of water-reducing admixture which are suitable, for example, for concreting work in hot weather or in a tropical climate. Retarders as such are rarely available, but retarding grades of plasticizer are used for delaying the set to facilitate finishing or to assist in the placing of large pours. They do not significantly affect the rate of gain of strength once setting has commenced and are in no sense substitutes for the use of low heat Portland cement.

2.4.3 **Air-entraining admixtures**

Air-entrained concrete is more durable than non-air-entrained concrete under the action of frost and the de-icing salts and fluids which are used on roads and airfield pavements in winter. The air bubbles have a plasticizing effect on the mix, which usually necessitates some minor changes in mix proportions. An air content of 5 per cent by volume is considered optimum for concrete with 20 mm (¾ in) aggregate, and this is normally achieved with the addition of about 0·1 per cent air-entraining admixture by weight of cement. For other aggregate sizes, optimum air contents are given in Section 4.2.2. Each 1 per cent addition of air can reduce the potential strength by 4 to 7 per cent in a typical concrete, but reduction in the water content and extra cement in the mix can compensate for this drop. The increase of cement content is comparatively small since it is possible to lower the water content while maintaining a reasonable workability.

Most admixtures marketed as air-entraining agents are based on neutralized hydroxylated wood resin. In most cases this is sold to formulators under the trade name 'Vinsol resin' (very insoluble resin). To make it soluble, it is turned into soap by chemically reacting it with caustic soda to produce a dark brown liquid which tends to form a foam on vigorous shaking. Vinsol resin is chosen because it is relatively inexpensive, entrains bubbles of the optimum size (50 μm) and is consistent in its performance. There are several alternatives on the market which are based on synthetic or natural materials.

2.4.4 **Pigments**

Although experiments have been carried out on the production of coloured clinkers during the cement manufacturing process, the only practical way of preparing coloured concrete at the moment is to add a colouring agent to the concrete during mixing. Either grey or white Portland cement serves as the base, the latter giving brighter colours.

Pigments based on iron oxides are available in powder form in a range of colours: red, brown, yellow and black. They are inexpensive, colour-fast to both light and the alkalis in cement, temperature-stable (to varying degrees), and they have good tinctorial strength. An addition of 4 per cent iron oxide powder by weight of cement is normally sufficient to impart a satisfactory tint to a concrete or mortar mix. Other pigmenting materials include carbon black, green chromium oxide, titanium white and cobalt blue as well as two organic green and blue dyes.

If large quantities of pigment are used, there may be an effect on the workability of the mix. In most cases this effect is small, but problems with workability and strength may occur when trying to produce a dense black with carbon black. Care must be taken with the batching of pigments if uniformity of colour is to be achieved.

2.4.5 **Water-repelling admixtures**

Structural concrete which is designed to resist cracking and is made from dense concrete cast in thick enough sections will be virtually impermeable to the flow of water, even under pressure. However, chemicals are sometimes employed as water-repelling admixtures. These are especially useful when the concrete has a porous texture, as, for example, with cast stone, or when the section is thin, as in a rendering for lining a leaky basement. Admixtures, however, can only marginally improve the impermeability of concrete.

The usual chemical for this application is calcium stearate, a metallic soap supplied in powder form which has a repelling action on water and is compatible with cement. When incorporated in a mix, it tends to form a lining on the capillary and larger pores in the concrete, imparting an electrical charge which effectively stops the movement of liquid water within the pore system.

A further use of water-repelling admixtures is to reduce

the need for frequent cleaning of buildings which would otherwise be necessary when rain has washed dirt into the surface. Water-repellent cement is sometimes used for the same purpose.

These are available a number of proprietary water-proofing systems which are based on cement. These often include calcium chloride in the formulation which, at high dosage rates, acts as a quick-setting admixture. Sometimes a rubber latex dispersion is also included to act as a bonding and permeability-reducing agent.

2.4.6 Pumping aids

When concrete is placed by pump, the applied pressure tends to make the water in the mix flow through the solids. Mix design for pumping is aimed at minimizing this, but it is sometimes useful to add a material that has the property of thickening the water phase. As with waterproofers, the effect of a thickener (such as polyethylene oxide, which is the usual material for this purpose) is only marginal. Some cellulose derivatives which may be used also cause air entrainment.

When pumping a very rich concrete mix with a cement content over 400kg/cm^3 (670 lb/yd^3), a reduction in viscosity of the water may be desirable. In these circumstances a simple water-reducing plasticizer may prove beneficial.

When a problem with the pumping of concrete occurs, it is important to establish the nature of the difficulty before selecting an admixture to improve the situation.

2.4.7 Pozzolanas

Pozzolanas are materials which possess cementing properties in the presence of lime, including the lime derived from Portland cement. The name arises because there is a well-known deposit of volcanic material having this property near the town of Pozzuoli in Italy. Natural pozzolanas occur in many parts of the world, but in Britain the only significant materials of this type are ground granulated blastfurnace slag and pulverized-fuel ash, more commonly referred to as PFA or fly-ash, from coal-burning power stations.

Not all PFA is suitable for use in concrete, so care is needed when choosing the source of supply.

The rate of gain of strength of pozzolanic mixes is fairly slow, with a corresponding slow rate of heat evolution. Pozzolanas have thus been used as a partial substitute for cement in situations, such as dams and other massive structures, where it is necessary to reduce the amount of heat evolved or the rate at which it is liberated.

2.4.8 Dispensing of admixtures

The use of admixtures necessitates a greater degree of control than normally applies to the other mix constituents. Most admixtures have to be dispensed in small doses which are susceptible to proportionately large errors. Mixing must be thorough, and evenness of colour, which is normally a good guide to uniformity of mixing of concrete, does not necessarily give a guide to uniform dispersion of an admixture.

2.4.9 Storage of admixtures

Admixtures are usually delivered to site in steel or plastics drums. These should be kept indoors since the quantity required at any one time is usually small. Admixtures can degrade at extremes of temperature, causing their physical properties to change and making accurate dispensing difficult.

3 CONCRETE MIXES

Two essential properties of hardened concrete are durability and strength. Both properties are affected by the voids or capillaries in the concrete which are caused by incomplete compaction or by excessive water in the mix. Within certain limits, the higher the cement content and the lower the water/cement ratio, the stronger and more durable will be the concrete. Dense, impervious concrete is also necessary if it is to retain or exclude water and provide corrosion protection to reinforcement.

The requirement that air voids be kept to a minimum means that the materials must be so proportioned that the mix is workable enough to be fully compacted with the means available. The use of mechanical compaction equipment allows a drier and potentially stronger and more durable mix to be used than if compaction is done by manual methods. However, except for very high strengths, mixes can be designed that fulfil strength requirements yet still allow the concrete to be compacted by hand, though the cement contents will be higher.

3.1 Specifications

Specifications vary widely, depending on their source. Generally concretes are divided into two classifications. The first comprises 'designed mixes', where strength is the main criterion specified, the design of the mix is left to the supplier and compliance is judged on the basis of strength testing. A minimum cement content is also specified to produce the required durability. The second classification is that of 'prescribed mixes', where the cement content or mix proportions are specified and it is the duty of the specifier to ensure that the mix specified will give the required properties, including strength. With prescribed mixes, strength tests are not used to judge compliance with the specification.

3.2 Workability

The term 'workability' is used to describe the ease with which the concrete can be compacted. The cement content, the overall grading of the aggregate and the shape of the aggregate particles affect the amount of water required to produce 'workable' concrete. The workability may be measured and defined by the standard tests described in Section 10.

3.3 Strength

The strength of concrete is usually defined by the crushing strength of 150 mm (6 in) cubes at an age of 28 days. However, other types and ages of test and other sizes and shapes of specimen are sometimes used; for example, highway contracts frequently make use of indirect tensile tests conducted on concrete cylinders. Test procedures are described in Section 10. The strength will probably be specified as a characteristic strength. This is the strength below which not more than a stated proportion of test results fall. The variation in results needs to be considered statistically, and a detailed discussion on this subject is outside the scope of this paper. However, it will be briefly mentioned to clarify the consideration of concrete strengths.

Because of the variability of test results, the concrete must be designed to have an average strength greater than the required characteristic strength, otherwise more than the permitted percentage of results would fall below the characteristic strength. The difference between the average and characteristic strengths is known as the 'margin'. The spread

of results from concrete strength tests has been found to follow what is known, in statistics, as a 'normal' distribution, which enables it to be defined by the 'standard deviation' of the results. Where the spread and the standard deviation are large, the margin also must be large, but where site control over materials, mixing and testing procedures is good, the standard deviation will be smaller and the margin may be reduced, leading to economies in materials.

There is a mathematical relationship between the margin, the standard deviation and the proportion of results falling below the characteristic strength. If 5 per cent of results fall below, then the margin will be 1·645 times the standard deviation.

In order to use this statistical method reliably a large number of test results is needed. Yet compliance with the specification is commonly judged by examining smaller numbers of results. The effect of this is to introduce a risk for the supplier that some of his concrete may fail to meet the compliance requirements even though its strength agrees with the definition of the characteristic strength. Care must, therefore, be taken to choose a suitable margin when designing a concrete mix. During a long contract the materials will vary, and by keeping continuous records of test results it is possible to vary the margin so as to make the best use of the materials while complying with the specification; the cement content must not, however, be reduced below the specified minimum figure.

3.4 Grades of concrete

A limited range of concrete grades designed by a number equal to the characteristic strength in N/mm^2 is given in Table 2.

TABLE 2

RECOMMENDED GRADES OF CONCRETE

Grade	Characteristic strength N/mm^2	Characteristic strength lbf/in^2*	Lowest grade for compliance with appropriate use
7 10	7·0 10·0	1000 1450	plain concrete
15	15·0	2200	reinforced concrete with lightweight aggregate
20 25	20·0 25·0	2900 3600	reinforced concrete with dense aggregate
30	30·0	4350	concrete with post-tensioned tendons
40 50 60	40·0 50·0 60·0	5800 7250 8700	concrete with pre-tensioned tendons

* The values in imperial units are approximate and are included for comparison only. They should not be used for purposes of mix design or for the interpretation of test results.

3.5 Design of mixes

Once an average strength has been decided upon, the mix must be designed to meet this and any other requirements, including workability. There are a number of methods that can be used to arrive at the optimum proportions, but they are all approximate. All accurate mix design requires a trial mix, the proportions of which may need modification in the light of experience. This approach, of using a theoretical method to arrive at a first approximation followed by modification by trial mixes to achieve a practical end result, is inevitable when dealing with natural materials.

Mix design methods are described in several publications and the subject will not be dealt with in any great detail here.

3.6 Prescribed mixes and standard mixes

Concrete mixes may be specified to have fixed proportions of the constituents by weight. The 'grade' is the minimum characteristic strength which the concrete may be expected to have although, due to the variability of materials throughout the country, most concretes will have strengths comfortably in excess of this figure. It should be emphasized, however, that strength is not used as a criterion by which to judge acceptance of prescribed mixes.

3.7 Nominal mixes

Following the practice first established many years ago, concrete mixes are still sometimes specified in terms of volumetric proportions. This practice is dying out for structural concrete, though some Codes of Practice still allow concrete to be specified in this way. Nominal mixes are sometimes specified to have minimum strengths in addition to the given proportions; in practice this is often interpreted, incorrectly, as a designed mix. The resulting concrete gives the required minimum strength but is often leaner than is specified; and it may then have too low a cement content to give the required durability.

Volume batching is difficult to control and nominal mix specifications provide no control over water content, so this type of specification should not be used unless site conditions make it necessary and it is certain that the concrete will be adequate for the work in hand. Weigh-batching of materials is preferable, and it is usually best, if the specification calls for nominal mixes, to agree weigh-batched alternatives with the engineer.

3.8 Effects of mix constituents

3.8.1 Cement

The effects of different types of cement have been noted in Section 2. Within one type the properties will vary, but if the supply is derived from one works only, this variation will be small.

3.8.2 Aggregate

Of the materials in concrete, aggregate is the most variable. The overall grading of the aggregate affects the amount of water that must be added because 'fine' gradings require more water than 'coarse' gradings to obtain the same degree of workability. The special considerations applicable to air-entrained concrete are discussed in Section 4.2.2. Aggregate particles which have sharp edges or a rough surface, such as crushed stone, need more water than smooth and rounded particles to produce concrete of the same workability. It may be necessary to increase the cement content of a mix made with crushed aggregate or irregularly shaped gravels to allow water to be added in order to make the concrete sufficiently workable without reducing the strength below the required level. However, due to interlock between aggregate particles, a crushed aggregate concrete may have a higher strength than a smooth or rounded aggregate concrete with the same water/cement ratio, and this extra strength may be sufficient to offset the effect of the extra water. This is particularly so at high strength levels.

As the maximum size of the aggregate is reduced, the cement content of the mix will need to be increased to give the same workability with the same water/cement ratio. This is because the surface area of aggregate to be wetted is greater with the smaller aggregate size.

The fine and coarse aggregates should be proportioned to obtain the required workability with the minimum amount of water. Badly proportioned constituents require an excessive amount of water to give adequate workability, and this will result in concrete of low strength and poor durability.

3.8.3 **Water**

Variation in water quality or quantity should be avoided. Water is the most consistent of the constituents of concrete but water quantity, and in particular the water/cement ratio, is most important for the production of concrete of consistent strength. The amount of water used should be the minimum necessary to give sufficient workability for full compaction of the concrete. When deciding how much water to use, allowance must be made for absorption by dry or porous aggregates and for the free surface moisture of wet aggregates. For example, if the free water/cement ratio is given as 0·45 by weight, the total quantity of water per 50 kg (110 lb) bag of cement should be 0·45×50=22·5 litres (5 gal), including the free water contained in the aggregate.

3.8.4 **Admixtures**

Admixtures have been described in Section 2. Some, particularly lignin-based plasticizers, are natural products and are subject to variation. All admixtures batched in small quantities need care in dispensing and mixing.

3.9 **Batching**

The practice of batching and mixing is described in Section 5. It is useful at this point, however, to consider general principles. Weigh-batching is preferable in most cases, though in some circumstances lightweight aggregates may be measured by volume. Cement should always be measured by weight because its bulk volume is affected by the compaction it receives in handling. It is now usual to specify all materials by weight because, with properly maintained plant, batching by weight is more accurate than batching by volume.

3.10 **Trial mixes**

For designed mixes, trial mixes should be prepared by the supplier for all concrete except grades 7 and 10 and, in the case of dense aggregate, grade 15, unless there are existing data showing that the proposed mix proportions will produce a concrete that is of the grade required and has adequate workability for full compaction by the method to be used during placing.

Tests at ages earlier than 28 days are usually performed at 3 or 7 days. Alternatively, special accelerated cube testing routines may be used which give an estimate of the 28-day strength at about 24 hours after mixing.

4 CONCRETE PROPERTIES

The properties of concrete are too many and varied to make it either possible or necessary to consider them in depth in this paper; therefore, only strength, durability and deformation are considered here, abrasion resistance and other properties, which may be essential in some circumstances, have been omitted.

4.1 **Strength of concrete**

4.1.1 **Characteristic strength**

Characteristic strength is the value of the strength of concrete below which not more than 5 per cent of the test results fall. The characteristic strength of the concrete should be specified at one age only. Unless specified otherwise, the strength tests should be carried out at an age of 28 days for concrete made with a Portland-type cement.

For control purposes, testing may be carried out at an earlier age to determine whether the mix is likely to attain the required 28-day strength, thus making is possible to alter the mix proportions within a shorter period of time. Only rarely is an age different from 28 days used for judging compliance with a specification.

4.1.2 **Flexural and indirect tensile strength**

Where the flexural or the indirect tensile strength of concrete is a critical property, the specification and control may be based on that property. Tests for measuring these properties are described in Section 10. The tensile strength of concrete is generally taken to be about one-tenth of its compressive strength, but different aggregates cause large variations in this proportion and a compressive test is only a very general guide to the tensile strength.

4.1.3 **Increase in strength with age**

Owing to continuing hydration of the cement, the strength of concrete increases with age. For concrete made with ordinary Portland cement an increase of strength to be used in design is allowed, as given in Table 3.

The variation of strength with age in any particular case will not agree exactly with the figures given in Table 3, which are based on average behaviour. The source of cement and the curing conditions are influential factors in the actual gain of strength, particularly at early ages. Where the thickness of the concrete is small, or in other conditions where the concrete may dry out quickly, there may be little gain in strength with age. Higher gains in strength may be found where the concrete is kept permanently wet or where the volume of concrete is large enough to prevent drying out within the mass.

For lightweight aggregate concrete the increase in strength with age is generally as given in Table 3, though it should be noted that a characteristic strength of 50 N/mm^2 is difficult to achieve. It is in any case advisable to check with the aggregate producer when rich mixes are being used, because in some cases there may not be a significant increase in strength after 28 days.

4.2 **Durability of concrete**

The resistance of concrete to weathering, chemical attack, abrasion, frost and fire depends largely upon its quality and constituent materials. In reinforced and prestressed concrete, protection of the steel against corrosion is influenced by the depth of cover provided and the permeability of the concrete. In some environments, appreciable deterioration of unprotected concrete is inevitable, in other environments, adequate durability can be achieved by using a good-quality concrete and an appropriate type of cement. The crushing strength alone is not a reliable guide to the quality and durability of concrete; it must also have an adequate cement content and a low water/cement ratio. Furthermore, the mix must be workable so that it can be fully compacted to produce a dense,

impermeable concrete and provide the required finish.

4.2.1 **Permeability**

Low permeability of concrete is important in increasing resistance to frost action and chemical attack and in protecting embedded steel against corrosion. To produce concrete of low permeability, full compaction and proper curing is essential. For given aggregates, the permeability of concrete can be reduced by reducing the water content or by increasing the cement content. In practice, however, there is a limit to the reduction which can be made in the water content if full compaction is to be achieved.

4.2.2 **Frost resistance**

The frost resistance of concrete depends on its permeability and the degree of saturation of the concrete when exposed to frost; concrete with a higher degree of saturation is more liable to damage.

The use of salt for de-icing roads greatly increases the risk of frost damage. Air-entrained concrete has high resistance to frost attack and should be used where concrete is exposed to salt used for de-icing, e.g. in roadside structures. The average air content of the concrete should be: —

for 10 mm ($\frac{3}{8}$ in) nominal maximum size aggregate, 7%
for 14 mm ($\frac{1}{2}$ in) nominal maximum size aggregate, 6%
for 20 mm ($\frac{3}{4}$ in) nominal maximum size aggregate, 5%
for 40 mm ($1\frac{1}{2}$ in) nominal maximum size aggregate, 4%

A tolerance of $\pm 1\frac{1}{2}$ per cent is usually permitted on the average values given above. Air-entraining admixtures are described in Section 2.4.3.

4.2.3 **Resistance to chemical attack**

Portland cement concrete is attacked by acids and by acid fumes, including organic acids which are often produced when foodstuffs are being processed. Vinegar, fruit juices, sour milk and sugar solutions all attack concrete. Alkalis have little effect. The most common forms of chemical attack which concrete has to resist are the effects of sulphates in the soil and in sea water.

The resistance of concrete to attack by sulphates and sea water depends on the type of cement used, the cement content and the water/cement ratio. The requirements to meet specific conditions of sulphate attack are described in Table 1.

In all cases where concrete is subject to chemical attack, resistance is related to the cement content, the water/cement ratio and the degree of compaction. Attack on a dense, impermeable, well-compacted concrete will be slow, and a well-compacted concrete with a high content of ordinary Portland cement may be more resistant to sulphates than a poorly compacted concrete with a lower content of sulphate-resisting Portland cement.

4.2.4 **Resistance to fire**

The resistance of concrete to fire is related to the type of aggregate, the concrete strength and the cover to the reinforcement. A test on an element of construction will provide conclusive evidence of its performance and decide its fire-resistance grading. The fire resistance of certain standard types of reinforced concrete construction is given in the relevant standards and from this information dimensions can be chosen to provide an adequate degree of fire resistance.

4.2.5 **Maintenance and protection**

Generally maintenance is not needed for concrete construction. Where, however, the concrete is exposed to attack by weather or chemical action maintenance may be needed. Protective coatings will delay or prevent deterioration of the concrete in such cases. The protective coating to be used will depend upon the particular form of exposure, but it should be durable and able to adjust itself to elastic and thermal movements of the structure. All protective coatings should be maintained in good condition by renewed application during the life of the structure.

Any concrete paint should be suitable for the alkaline character of concrete. Suitable types of paint and the precautions to be taken in their use are available.

A periodical check should be made (e.g. every three or five years) to detect any excessive cracking or other defect of the concrete.

Where corrosion of the bars has caused staining or has loosened the concrete cover, the life of the structure can be considerably prolonged by exposing, cleaning and recovering the bars.

4.3 **Deformation of hardened concrete**

The principal components of the deformation of hardened concrete are: —

(1) elastic deformation, which takes places instantaneously and is dependent on the applied stress;

(2) drying shrinkage, which occurs over a long period and is independent of the stress in the concrete;

(3) creep, which also occurs over a long period but is dependent on the stress in the concrete.

In addition, concrete, like any other material, is subject to expansion or contraction when the temperature changes. The numerical values for elastic deformation, shrinkage and creep are dependent on a number of factors which include the effects

TABLE 3

INCREASE IN STRENGTH WITH AGE

Grade	Characteristic strength		Cube strength at an age of 7 days		2 months		3 months		6 months		1 year	
	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²
20	20·0	2900	13·5	1960	22	3190	23	3340	24	3480	25	3600
25	25·0	3600	16·5	2390	27·5	3980	29	4200	30	4350	31	4500
30	30·0	4350	20	2900	33	4780	35	5070	36	5220	37	5360
40	40·0	5800	28	4060	44	6370	45·5	6600	47·5	6880	50	7250
50	50·0	7250	36	5220	54	7830	55·5	8050	57·5	8330	60	8700

* The values in imperial units are approximate and are included for comparison only. They should not be used for purposes of mix design or for the interpretation of test results.

of the environment, the age of the concrete, its mix proportions and its contituent materials.

4.3.1 **Elastic deformation**

The modulus of elasticity is primarily related to the crushing strength of the concrete. It is, however, influenced by the elastic properties of the aggregate and to a lesser extent by the conditions of curing, the age of the concrete, the mix proportions and the type of cement. The values of modulus of elasticity related to strength for use in design, are shown in Table 4.

TABLE 4

VALUES OF MODULUS OF ELASTICITY OF CONCRETE FOR USE IN DESIGN

Cube strength of concrete at the appropriate age or stage considered		Modulus of elasticity of concrete	
N/mm²	lbf/in²	kN/mm²	lbf/in²
20	2900	25	$3{\cdot}6\times10^{-6}$
25	3600	26	$3{\cdot}8\times10^{-6}$
30	4350	28	$4{\cdot}1\times10^{-6}$
40	5800	31	$4{\cdot}5\times10^{-6}$
50	7250	34	$4{\cdot}9\times10^{-6}$
60	8700	36	$5{\cdot}2\times10^{-6}$

* The values of imperial units are approximate and are included for comparison only. The values should not be used for purposes of mix design or for the interpretation of test results.

4.3.2 **Poisson's ratio**

Poisson's ratio for the elastic strain under normal working stress may be taken as 0·2.

4.3.3 **Drying shrinkage**

The shrinkage of concrete is dependent on the amount of drying that can take place. It is therefore influenced by the humidity and temperature of the surrounding air, the rate of air flow over the surface and the proportion of surface area to volume of concrete. For a given environment, the shrinkage of concrete is influenced most by the total amount of water present in the concrete at the time of mixing, and to a lesser extent by the cement content. Another important factor is the shrinkage of the aggregate; in some parts of Scotland the aggregates can contribute significantly to the drying shrinkage of the concrete. The amount of reinforcement has a great influence on the shrinkage strain actually observed in a structure.

For small masses of concrete, drying out of doors in summer, it is reasonable to assume that half of the total shrinkage takes place in about one month and that about three-quarters of the total shrinkage takes place in six months. For a large mass of concrete, the shrinkage in one year may represent about half the total shrinkage, though this is dependent on the ambient conditions.

A typical value for the ultimate shrinkage strain in an unreinforced structure, drying to ambient conditions in Britain, would be about 500×10^{-6}, although wide variations are possible.

4.3.4 **Creep**

The creep of concrete is dependent on the stress in the concrete. For stresses of up to one-third of the cube strength it may be assumed that it is directly proportional to stress. Creep is influenced by the surrounding humidity and temperature, the cement content in the concrete, the water/cement ratio, the type of cement and the nature of the aggregate. The mass of the concrete has some influence on the amount of creep, but this is less than its effect on shrinkage; because there is some disagreement on the magnitude of the effect, it is recommended that it should be ignored until more information is available. The most useful parameter in assessing creep is the ratio of the applied stress to the cube strength of the concrete at that time.

Creep takes place over many months. In general, creep strain is proportional to the logarithm of the time under load.

If load is removed from concrete, an instantaneous elastic recovery occurs, and this is followed by a continuing expansion known as creep recovery. The creep recovery strain is usually about one-third of the original creep strain.

As with shrinkage, creep in a structure is restrained by the reinforcement present.

4.3.5 **Thermal expansion**

The thermal expansion of concrete is influenced by the type of aggregate, the water content of the concrete and the mix proportions. Where no information is available on the materials being used, the coefficient of thermal expansion of concrete at constant moisture content may be taken as $1{\cdot}0\times10^{-5}$ per °C. Where limestone aggregates are used, the value is likely to be less, usually about $0{\cdot}8\times10^{-5}$ per °C, and where limestone is used for both coarse and fine aggregate, the coefficient of expansion may be only about $0{\cdot}5\times10^{-5}$ per °C. On the other hand, for concrete made with aggregates rich in silica, such as most gravels, the value is likely to be about $1{\cdot}2\times10^{-5}$ per °C.

There is insufficient information available on the thermal expansion of lightweight concrete for a representative value to be quoted. Until such information is available, the same value should be used as for ordinary concrete.

4.4 **Early-age movements**

4.4.1 **Heat of hydration movement**

Besides the movements to which hardened concrete is susceptible, there are additional movements during the setting and hardening process. The most important of these is movement resulting from the heat of hydration of the cement. The setting reaction begins to evolve heat before the mix has stiffened. This causes thermal expansion of the plastic concrete which then sets in an expanded condition. When the concrete cools, contraction will occur which, if restrained, may lead to cracking.

In heavily reinforced concrete work the problem is usually confined to slabs over about 1 m (3 ft) thick and to walls. Walls are particularly susceptible because they are lightly reinforced horizontally and the timber form-work commonly used acts as a thermal insulator, thus encouraging a larger temperature rise. The problem may be reduced by a reduction in cement content, the use of a cement with a lower heat of hydration, the use of steel forms, a reduction in the placing temperature of the concrete and the use of small-diameter distribution reinforcement at centres less than 150 mm (6 in). Prevention of cracking in walls cast in timber forms cannot be guaranteed unless lengths and heights of pour are restricted to less than 3 m (10 ft). Such small pours, besides being uneconomic, give increased problems with construction joints and it is usually better to pour bay lengths of 8 to 10 m (25 to 35 ft) with crack inducers fixed to the form to produce an easily sealed groove. Crack inducement is helped if half the distribution steel is stopped at the section and if an inflated tube or similar section-reducer is included so that the section

is reduced to half the wall thickness. The hole so formed may subsequently be grouted. Crack-inducing water bars serve the same purpose.

A further factor to consider in wall construction is that panels tend to crack into areas which are nearly square. Thus a low wall may have a tendency to crack at closer centres than a higher wall, although the resulting cracks will be smaller.

4.4.2 **Plastic movement and bleeding**

Fresh concrete is a suspension of solids in water; when placed in the form, the solids tend to settle. This can result in 'bleeding', in which water appears on the surface. Bleeding can result in the top layers of a pour being weak and porous, a fault which may be prevented by careful selection of the fine aggregate, by a reduced water content in the mix or by using a drier mix to complete a pour.

If the settlement of solids is prevented by obstructions such as the top reinforcement in a slab or beam, cracks tend to form as the settling solids fold over the obstruction. These cracks frequently mirror the reinforcement arrangement. They should be closed to prevent corrosion of the steel. The best way of doing this is to delay the finishing of the surface as long as possible and to perform this operation as the mix finally stiffens, effectively preventing further settlement. If this is impossible, cracks that have formed may be sealed the day after pouring by brushing in dry cement.

An associated form of cracking in slabs occurs if water is allowed to evaporate before the concrete has set. The resulting reduction in volume of the top layer of concrete leads to cracks which can seem very wide, although they usually taper quickly and rarely penetrate through the slab. Absorption of water from the concrete by a porous formation can lead to a similar effect: this may be more serious because the cracks are more likely to run completely through the slab. This type of cracking may be prevented by thoroughly soaking the formation before concreting a ground slab or a foundation, or preferably by using a waterproof membrane. Surface cracks may be dealt with by the methods outlined above.

Curing measures designed to reduce drying shrinkage and eliminate surface cracking are described in Section 9.

5 SITE PRODUCTION

The series of operations which must be performed on the raw materials in order to turn them into the finished structure will, to a large extent, determine the strength, durability and appearance of the finished work. This sequence of operations is expensive, but carelessness and skimping will almost invariably prove a false economy.

5.1 **Access**

The need to provide for delivery of materials may impose limitations on the site. On most sites the heaviest and most frequent deliveries will be of aggregates and cement or ready-mixed concrete. Site roads capable of withstanding the expected traffic are always a good investment.

5.2 **Storing materials**

Materials must be stored so that no harm results during storage. Cement must be kept dry in silos, or, if in bags, it should be kept under cover and off the ground. Aggregates should be handled and stored so as to avoid segregation and contamination by other aggregates, fuel, mud, etc. Each site will impose its own conditions and will have to be considered individually. Factors to be taken into account are access, the storage area available in relation to the quantity to be stored, avoidance of double handling and convenience in relation to the remaining operations. Particular problems of storage are discussed in Section 2.

Storage of water is not usually a problem if the water supply is from a Water Board main. However, if water is abstracted from a stream or well, some storage may be needed in addition to the tank provided on the batching and mixing plant. The same is true if a small mixer not fitted with measuring equipment is being used.

5.3 **Batching and mixing**

Careful use of accurate measuring equipment is essential if good-quality concrete is to be produced. The accuracies required for measuring the different mix constituents are shown in Table 5.

Batching of cement should be by weight or by the use of a whole number of 50 kg (110 lb) bags. Batching of aggregates should normally be by weight, but with light-weight aggregates and with small batches volume batching may be desirable. If sand is batched by volume, allowance must be made for 'bulking', an increase in volume which occurs when the sand is moist. This is considered in Section 10.2.2.4. Water may be batched by weight or by volume. Solid admixtures should be batched by weight, whereas liquid or paste admixtures may be batched either by volume or by weight.

Measuring equipment should be set up on a firm, level base and hoppers should be loaded evenly. The equipment should be checked immediately after being set up and should thereafter be checked for accuracy at regular intervals, as recommended by the manufacturer.

TABLE 5

RECOMMENDED BATCHING MARGINS

Material	Accuracy of measurement
aggregates cement water	±3% of batch quantity
admixtures	±5% of batch quantity

5.3.1 **Moisture content of aggregates**

The water contained in the aggregates can be a significant proportion of the required water content of the mix. It is therefore necessary to know how much water is being carried into the mix by the aggregates, especially if the batching accuracies given in Table 5 are to be achieved. If the weather is settled and the requirements for storage given in Section 5.1 have been observed, then the water content of the aggregates need not be measured more often than once per day; but if loads of aggregates are to be used immediately after delivery their water content should be checked prior to mixing. Available methods include the determination of loss in weight on drying, use of a 'Speedy' moisture meter and several displacement methods. These are described in Section 10.2.2.4.

5.3.2 **Concrete mixers**

Concrete mixers are designated by a number representing the nominal batch capacity in litres and the letter indicating

the type of mixer, as follows: —

(1) Tilting drum, type T

(2) Non-tilting drum, type NT

(3) Reversing drum, type R

(4) Forced action, type P. The forced action type is commonly known as a pan mixer.

Thus a 200 litre (7 ft^3) tilting drum mixer is designated as 200T.

5.3.3 **Operation of mixers**

Most mixers are fitted with power-operated loading hoppers and automatic or semi-automatic water.measuring devices. Some mixers also have admixture dispensers fitted. It is normally best to add the water at the same time as the other materials to ensure its even distribution.

When the concrete is mixed the complete contents of the drum should be discharged in one operation; an intermediate wet hopper may be used if it is not possible to remove all the contents from the area of the mixer immediately. If this is not done there is a risk that the larger aggregate may separate from the mix, so that different container loads will have different properties. At the start of the day the batches will be harsh and stony because some mortar will stick to the inside of the drum and around the blades. To counteract this, the proportion of coarse aggregate should be reduced for the first batch or two.

After use, the mixer should be thoroughly washed out and the blades cleaned to prevent hardened concrete building up around the drum and blades. The inside of the drum should be inspected regularly and any blades which are worn or broken should be replaced. Inspection and maintenance are necessary to avoid reduction in efficiency and loss of time due to a breakdown.

5.3.4 **Mixing time**

Thorough mixing of concrete is essential. Mixing times will vary according to the mix, the mixer and whether or not the mixer is filled to capacity. It is important to realize that the makers' quoted mixing times are not always applicable in all conditions. A uniform colour is usually a good guide to efficient mixing.

5.3.5 **Ready-mixed concrete**

Ready-mixed concrete is used on many sites. In addition, some sites use a central batching plant and employ transit mixers, owned by the contractor, to mix and transport concrete. It is important that these transit mixing trucks be operated according to the manufacturer's recommendations. If the drum speed is too high or too low, inefficient mixing may result.

5.3.6 **Workability**

The workability of a concrete mix must be adequate to enable the concrete to be fully compacted with the available method of compaction. Depending on the degree of workability, it may be described by the results from a slump, a compacting factor or a 'Vebe' test. These tests are described in detail in Section 10. Very stiff mixes are best measured by the Vebe test, whereas workable mixes are best measured by the slump test: the compacting factor test is used for fairly stiff or intermediate workabilities. The slump test is the simplest and most often used.

None of these tests measure any basic properties of the concrete, but with any given set of materials they give a rough guide to the water content of the mix. Workability is normally judged by eye, with the test used to support the visual observations. In any case, a visual check of workability is a good guide to uniformity of concrete because it can be applied readily to each batch, whereas it is usually impracticable to apply a workability test to each batch.

5.4 **Transport**

A number of methods of transport are available, ranging from hand wheelbarrows to concrete pumps. The chosen method will depend on the size and complexity of the site and such factors as whether or not a crane is available. In all cases the concrete must be transported so that it does not segregate and so that it does not become contaminated with water or any other material after it has left the mixer. Where concrete is to be placed below the level of the supply, a chute should be considered because gravity is the cheapest means of transport, but it must be borne in mind that there is a danger of segregation.

5.4.1 **Pumping**

The transport and placing of concrete by pump is an increasingly popular method. It is very fast and efficient and results in little waste of concrete. Not all concrete will pump, and minor variations in the concrete mix are sufficient to make an otherwise pumpable mix completely unpumpable. The sand grading is particularly important and variations in grading can rapidly cause unpumpability. If only a small part of the mix in the hopper proves to be unpumpable the pump may become blocked, leading to a time-consuming and expensive delay while the pump is stripped down and the blockage removed. Consequently, great attention to detail is required in the design of the mix, the mixing process and the materials from which the concrete is made. The various available types of pump can differ radically in their design and power outputs, but there is not a very wide range of mixes which will be acceptable by one pump but not by another. In general, concrete is either pumpable or unpumpable.

A mix may fail to pump for one of two reasons. It can either disintegrate under the pressure imposed by the pump, in which case the aggregate particles lock together and arch across the pipe whilst the water-and-cement paste is squeezed out of the mix, or the pumping operation may fail because the friction against the pipe wall is too great for the pump to overcome. This latter case only occurs with very rich mixes or when a low-power pump is trying to pump a long distance. In all cases of difficulty with mixes which are known to be a little troublesome, pumping will be easier if a slow, steady rate of pumping is used, because both the risk of the mix disintegrating and the friction are decreased by slowing the rate of placing. Attempting to pump unpumpable concrete will only result in frustration and delay on the site, so any mix which looks doubtful or is markedly different from previous batches should be unhesitatingly rejected.

The speed of concrete pumping exceeds most other methods of placing by a significant margin, which is the real justification for the use of this expensive method. This fact should be borne in mind because it is very wasteful to use a machine capable of pumping 30 m^3 of concrete per hour if the placing gang can only place and compact 10 m^3 of concrete per hour. Concrete can be ruined by inadequate compaction by an overworked placing gang. Similarly, the supply of concrete to the pump should be adequate for the pump's capacity.

FIG. 1

Placing by pump. Note no further spreading of concrete is required.

The concrete pump is probably the most elaborate piece of machinery used on a building site. Accordingly, its maintenance requirements are considerable higher than those of other equipment. Manufacturers supply maintenance schedules and these should be rigidly adhered to. Worn parts should be replaced immediately. This is particularly necessary for pipes, because leaking pipes or joints can result in a blockage. Pumps should be grouted thoroughly before pumping begins and equally thoroughly cleaned out afterwards. Pipes should be cleaned after each use.

Some consideration should be given to the location of pumps on sites. If the concrete is to be supplied by ready-mixed concrete trucks, then ideally it should be possible for two trucks to discharge into the hopper of the pump at the same time so that one can be finishing its discharge as the second is starting, thus maintaining a continuous flow of concrete. If this is not feasible, then it should be possible for the ready-mixed concrete truck to leave the pump and a new truck to back in quickly to the hopper of the pump. On many sites the pump is working for a shorter time than it takes to change the trucks around —a considerable source of inefficiency.

Many pumps are now fitted with telescopic or lattice booms which are an advantage for placing concrete in difficult situations. However, a boom is not necessarily the best method of placing in all circumstances, and the matter should be approached with an open mind. There are circumstances where the conventional run of pipes is the best method, because a boom is a greater source of resistance to flow than a similar length of ordinary pipe of the same diameter.

5.4.2 Pneumatic placer

Pneumatic placers serve a function similar to concrete pumps, but the concrete is pushed through the pipeline by the direct application of compressed air to the concrete contained in a sealed hopper at the supply end of the pipe.

Most of the general considerations for pumping also apply to the use of a pneumatic placer, but there is the added danger of a plug of concrete being propelled by compressed air from the end of the pipe. For this reason it is common practice to fix a baffle or discharge box over the end of the pipeline.

5.4.3 Crane and skip

The crane is the most common method of handling concrete where vertical movement predominates. A crane is frequently needed on site for handling formwork and reinforcement, and its further use in transporting concrete to the point of placing may be both convenient and economic. However, if concreting requirements dominate the choice of crane capacity, particularly with large skips at maximum radius, it may be more economic to restrict the use of the crane to other materials and to transport concrete by other means. The crane varies in its rate of handling concrete, depending on the amount of winding and slewing needed, and the rate at which concrete can be placed is likely to be less than with some other means.

A poker vibrator may be used to assist a stiff mix out of a skip, but care is needed to avoid segregation. The rate at which the skip empties is sometimes a useful guide to consistence because a change in mix proportions will alter the rate of flow. Concrete may need to be handled twice if the skip deposits it in a heap which then needs to be distributed manually. It is best, especially when working on slabs, to empty the skip in a series of heaps, each sufficient for the immediate area to be placed.

5.4.4 Belt conveyor

Belt conveyors are much used in the United States, but little here. Their main purpose is for horizontal transport because they can normally be used only on slopes not exceeding 15°. However, in conjunction with some method of vertical transport, where necessary, they can be a useful means of distributing concrete at a fast rate.

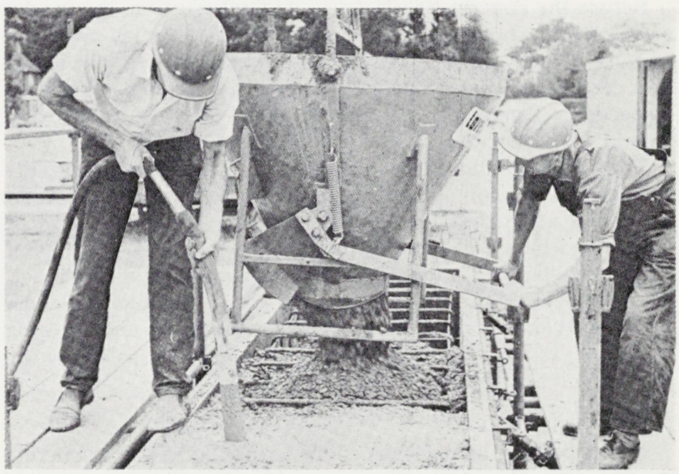

FIG. 2

Placing concrete from a skip and compacting by means of a poker vibrator.

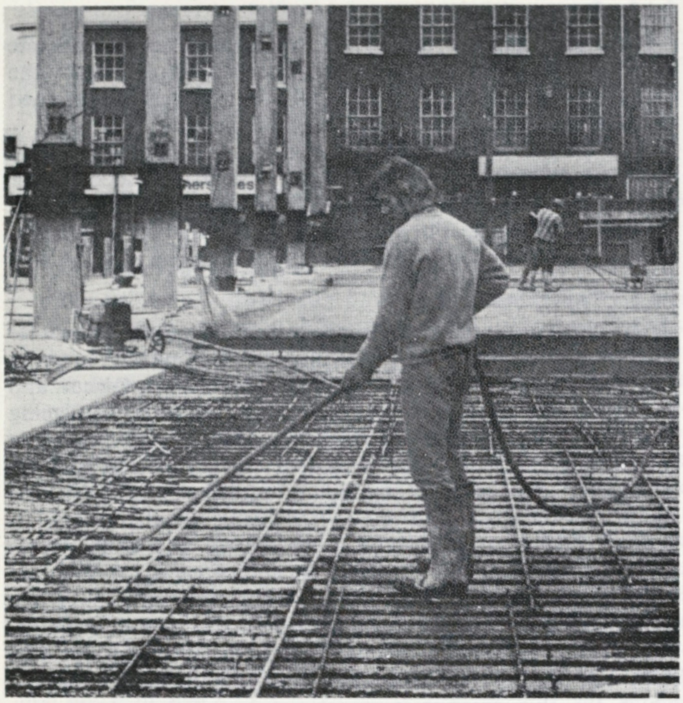

FIG. 3

Blowing out rubbish with a compressed air line.

5.4.5 **Other methods**

There are many other methods of transporting concrete, ranging from wheelbarrow and barrow hoist to dumper trucks, monorails and the railcars sometimes used in tunnel work. Wheeled vehicles can cause segregation on a long haul, and agitator trucks may be preferred to dumpers. However, there are few problems with these methods of handling and the choice will be determined by the balance of labour and plant costs and by the size of the job.

5.5 **Placing and compacting**

Before the concrete is placed in its final position, the insides of the forms should be inspected to make sure they are clean and have been treated with release agent. Where the forms are deep, temporary openings should be provided for this inspection. Rubbish, such as sawdust, shavings and wire, should be blown out with compressed air (Fig. 3).

The concrete should be placed in its final position rapidly so that it is not too stiff to work. On no account should water be added after the concrete has left the mixer. A heap of concrete, which will have to be moved to some other part of the form, should not be allowed to accumulate in one place. The concrete should be placed as closely as possible to its final position. It should never be moved by vibrating it and allowing it to flow, as this may result in segregation which will show on the surface of the finished work.

5.5.1 **Deep lifts**

It is becoming increasingly more economic to place concrete in deep lifts. The technique not only saves time but reduces the number of horizontal joints, which are often difficult to construct so as to be visually acceptable.

For placing in deep lifts to be successful, the mix must be designed to have a low risk of segregation and bleeding. The concrete should be introduced to the form through trunking, as this reduces impact damage to forms and reinforcement and enables the layer of concrete to be built up evenly. Trunking also prevents grout being left on the forms and reinforcement above the concrete level, resulting in a 'hungry' concrete at the bottom of the lift. The placing hose of a pump forms a useful 'trunk'.

5.5.2 **Vibration**

If concrete is to achieve its maximum strength, it must be compacted so that it contains the minimum of unwanted air. This is easy with a wet, workable mix, but the excess water in this sort of concrete contributes to weakness. The strongest and most durable concrete is as dry as can be fully compacted by the means available. Thus vibrators, which enable a drier mix to be completely compacted, have replaced the earlier hand methods and have led to the production of better concrete.

The most common type of vibrator is the internal vibrator or poker (Fig. 2). This is a vibrating tube at the end of a flexible drive. Pokers vary in size, usually from 25 to 75 mm (1 to 3 in) in diameter. A poker vibrator should not be dragged through the concrete, nor used to help heaps of concrete to spread out. It should be placed vertically in the concrete, held in position until air bubbles cease to come to the surface, then *slowly* withdrawn so that concrete can flow into the space previously occupied by the poker. This should be repeated at about 0·5 m (18 in) centres. The concrete should be placed in layers never more than 600 mm (2 ft) thick, and the vibrator should be lowered at least 100 mm (4 in) into the layer beneath. External formwork vibrators are occasionally used, but their usefulness is limited by the heavy formwork needed to resist the stresses and shaking they produce. They are often necessary, however, for heavily reinforced walls and the webs of deep beams, where it is difficult or impossible to insert an internal vibrator.

Slabs are best compacted by vibrating tamping beams. These combine the action of a screed and a vibrator, but they are only effective for a limited depth. In general, a slab more than 150 mm (6 in) thick should be compacted with a poker vibrator and finished with a vibrating beam, but the beam alone should be adequate for thinner slabs.

Over-vibration is to be guarded against, but it is not as common as under-vibration. Over-vibration may cause segregation and the formation of a weak layer of laitance at the top of a pour. To some extent this is determined by the mix—over-sanded, wet mixes tend to segregate. These same mixes tend to bleed, again producing weak concrete at the top of the lift. These faults may be corrected by varying the mix proportions.

5.6 **Construction joints**

Generally it is not possible to place concrete continuously; construction joints must therefore be provided. Such joints are a potential source of weakness. They should be located and formed with care, and their number kept to a minimum.

Wherever practicable, joints should be either vertical or horizontal. Vertical joints should be formed against a stop board. Expanded metal may sometimes be used for this purpose and left cast in; it is not, however, suitable for all cases because it may eventually corrode. Horizontal joints should be level and, wherever possible, be so arranged that the joint lines coincide with the architectural features of the finished work. Battens may be nailed to the formwork to ensure a

horizontal line and, if desired, may be used to form a grooved joint.

5.6.1 **Location of construction joints**

Wherever possible, the position of construction joints should be settled before any concreting begins. As a general rule, joints in columns are made as near as possible to the beam haunching. Joints in beams and slabs should normally be made at the centre, or within the middle third of the span. Horizontal joints in walls are usually made in positions such as the top of a plinth or the top or bottom of a window opening. Vertical joints in walls should be kept to a minimum.

5.6.2 **Preparation of construction joints**

As concreting proceeds, water sometimes collects on horizontal surfaces. If this occurs, a drier mix should be used for the top lift of the pour to avoid the formation of laitance—the layer of water, cement and very fine material from the aggregate—which prevents good bond between the old and the new concrete. Any laitance so formed should be removed by spraying the surface with water and brushing it to expose the coarse aggregate. Preferably this should be done an hour or so after the concrete has been placed. If the concrete is left overnight it may be necessary to use a stiff brush to clean the surface, but if the concrete has been allowed to harden it will be necessary to hack the whole of the surface, taking care to avoid damaging the aggregate. However, the best joints are obtained by light brushing soon after pouring.

5.6.3 **Water bars**

Water bars are often installed across construction joints to provide a positive barrier against the movement of water through the joint. They most commonly consist of a strip of rubber or plastic with a cross-section specially shaped to provide bond within the concrete on either side of the joint and to give sufficient flexibility to accommodate any expected movement of the joint. Water bars may be external to the concrete, in which case they are placed on the ground or fixed to the forms, or internal, requiring careful fixing in the stop-end form.

Where no movement is expected, for example at a horizontal construction joint in a wall, the water bar may be rigid. A mild steel strip is sometimes used.

Great care is needed when placing concrete around water bars because the space is often congested. If the concrete is poorly compacted and honeycombed, water can pass round the water bar and its object is defeated. It is also easy to damage or displace water bars if they are not rigidly held in position and insufficient care is taken during placing.

5.6.4 **Concreting at construction joints**

At a horizontal construction joint, curing of the joint surface should be suspended a few hours before concreting is to be resumed in order to ensure superficial drying. Sprayed curing membranes and release agents should not be allowed to contaminate the joint surface.

Just before concreting is resumed, the roughened joint surface should be thoroughly cleaned (without re-wetting) and loose matter removed. Special care should be taken to obtain thorough compaction and to avoid segregation of the fresh concrete at the joint plane. Rarely is it desirable to place a layer of mortar or grout against the joint, or to use a richer and more workable concrete in this position. Forms should be tight at this point because grout leaks cause weakness of the joint as well as being unsightly.

5.7 **Concreting in cold weather**

It is well known that water expands when it freezes. This can cause serious disruption if freezing is allowed to occur within partly set concrete. The setting and gain of strength of concrete are delayed at low temperatures, so it is necessary to protect concrete against cold for some time after concreting.

Many of the precautions that can be taken to protect concrete from cold make use of the heat that concrete evolves as it sets. However, this is only effective if the concrete temperature is sufficiently high at the time of placing for the heat evolution to start rapidly. To this end the temperature of the concrete *when placed in the form* should never be less than 5°C (41°F). To achieve this, the temperature in the mixer or ready-mixed concrete truck should be at least 10°C (50°F) to allow for heat losses during placing. Some ready-mixed concrete plants can deliver at this temperature.

It is usually easiest to raise the temperature of the fresh concrete by heating the mixing water. Aggregates should be free from frost because it requires as much heat to melt the ice as to heat the same quantity of water from 0°C to 80°C—in other words, aggregates should be kept both dry and covered. Heated water should be added to the mixer before the cement so that its temperature will have been lowered by contact with mixer and aggregates. If this is not done there could be a flash set when hot water comes into contact with the cement.

Sometimes, in very cold weather, aggregates must be heated to achieve the desired concrete temperature. This may be done by injecting live steam or using hot air blowers, closed steam coils or electric heating mats. If live steam is used, the resulting increase in moisture content should be allowed for, but this method probably produces the most uniform heating.

If concrete with a sufficiently high initial temperature is prevented from losing heat to its surroundings, the heat evolved during setting will protect it from damage by frost. Thus formwork should be insulated (in this respect it should be noted that 19 mm ($\frac{3}{4}$ in) plywood has fairly good insulating properties on its own) and slabs should be covered with insulating quilts immediately after laying. Slabs are very prone to frost damage because of their large surface area which dissipates heat quickly. In addition, slabs are open to drying winds which can add to the effects of low temperatures.

Even though concrete has been protected from frost during its early setting, the subsequent slow gain of strength in cold weather needs to be allowed for. Longer periods will be required before forms are struck than are necessary in warmer weather. The gain of strength of concrete is related to its 'maturity', which is the product of the temperature above −10°C (at which temperature concrete does not gain strength at all) and the number of hours at that temperature. Before exposure to freezing temperatures, concrete should have attained sufficient maturity. If the average concrete temperature is known, the required maturity may be transferred to a required 'pre-hardening period'. This applies to structures that are supported, such as walls or beams that are propped. Longer periods will be needed for members that need to support their own weight.

It is sometimes useful to make extra test cubes and store these under the same conditions as the structural member to which they relate. These cubes can then be crushed before

deciding if it is safe to strike soffit forms to beams or slabs.

Further refinements may be made in site organization to help keep work going in winter. They may not always be applicable, but they should be considered because their cost is usually small in relation to the benefits of a smooth flow of work, a quicker end to the job and no idle labour. One refinement that might be considered is the total enclosure of the work area with, for instance, polythene sheet fixed to the scaffolding, and the use of space heaters within this enclosure. Rapid-hardening cement could be used, or an increase in the strength of the concrete could be contemplated, its extra cement content allowing forms to be struck earlier than the specified mix would allow.

Keeping weather records and planning with an eye to the weather forecast is necessary for efficient winter working. Records of maximum and minimum temperatures, together with a more continuous record during working hours, will help towards an assessment of maturity and formwork striking times. This assessment should take account of wind and cloud cover because the temperature of the concrete is the factor that matters and this is not always the same as the air temperature. On a windy, cloudless night concrete can be cooled below the air temperature. The weather forecast is freely available by telephone and is an invaluable guide to the planning of winter work. Frost can usually be predicted and precautions taken. Specifications frequently call for precautions to be taken at specified temperatures, depending on whether the temperature is rising or falling.

Besides the use of rapid-hardening cement, an accelerating admixture can be used if permitted by the specification, but the severe restrictions imposed on the use of the commonest accelerator, calcium chloride, should be strictly observed (*see* Section 2.4.1). Where an accelerating admixture is used, the handling time of the concrete should be kept to a minimum. Ideally the concrete should be placed within 15 minutes of mixing, otherwise there is a danger of premature setting.

5.8 Concreting in hot weather

The difficulties of concreting in hot weather are largely overcome by placing the concrete with sufficient speed and preventing it from drying out. Concrete sets more quickly when it is hot and, consequently, it should not be left standing in hot weather. If there is likely to be a delay in placing, the aggregates could be sprayed with water to cool them, but the increase in water content should be allowed for. Alternatively, or in addition, a retarding admixture may be used. Ready-mixed concrete which may be subject to delay should be kept agitating. It is rarely necessary to take more elaborate precautions in this country, but it is always necessary to place concrete in a smooth, continuous operation to avoid the formation of 'cold joints', which may occur if concrete has stiffened too much before the next layer is placed on it or next to it.

Curing of concrete is very important in hot weather. In preventing the concrete from drying out it must be realized that wind is as damaging as sun in this respect. Concrete should be protected from sun and wind as soon as it is placed. Cracking due to premature drying can occur before normal curing methods can be applied. These 'plastic cracks' are rarely of structural significance but look unsightly. Usually they have to be sealed to prevent ingress of water; the treatment is the same as for cracks caused by settlement (*see* Section 4.4.2). If plastic cracks do occur, this should be taken as an indication that protection is needed.

6 REINFORCEMENT

Reinforcement may consist of round steel bars, interwoven or welded mesh fabrics, or twisted or deformed bars.

6.1 Bar sizes and dimensions

The preferred nominal sizes of bars and wires are as given in Table 6. Where it is necessary to use a large bar, the recommended size is 50 mm (2 in).

TABLE 6

PREFERRED NOMINAL SIZES OR REINFORCEMENT

Nominal size	mm	6	8	10	12	16	20	25	32	40
of bars	in	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$
Nominal size	mm	5	6	7	8	10	12			
of wires	in	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{9}{32}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$			

Mesh fabrics made up from the wire sizes given in Table 6 are classified as square, structural, long, wrapping and carriageway fabrics.

The cutting and bending dimensions of reinforcement should be given and also the preferred form of bar schedule and referencing. Reinforcement must be bent on proper bar-bending machines and should be to the specified dimensions and within the allowable tolerances. Recommended radii for bends are specified, and these should be used unless otherwise detailed. For mild steel the inside radius should be twice, and for high-tensile steel thrice, the bar diameter. It is impossible to fix steel and give the correct cover of concrete (on which both the strength and the durability of the structure depend) if the bars have not been bent accurately.

Reinforcement should not be bent or straightened in a way that will injure or fracture the material. All bars should preferably be bent cold. However, when the temperature of the steel is below 5°C (41°F) special precautions may be necessary, such as a reduction in the speed of bending. Alternatively, the steel may be warmed to a temperature not exceeding 100°C (212°F). Bars bent hot should not be cooled by quenching. Cold-worked reinforcement should never be bent hot.

6.2 Handling and storage

On the site, bars should be stacked off the ground and in such a way that it is easy for the bar bender to find the sizes and lengths he requires. The steel grade marks or identification tags attached to the bars by the manufacturer should always be clearly visible. When bars have been bent ready for fixing, each bundle should be clearly marked so that the steel fixer has no difficulty in selecting the correct bar.

Before concreting begins, the reinforcement should be free from mud, oil, paint, loose rust or scale, snow and ice or any other substance which will weaken its bond with the concrete. Normal handling prior to embedment in the concrete is usually sufficient to remove loose rust and scale from the reinforcement, but wire brushing may be necessary in some cases. If a retarding agent is applied to the forms to aid the production of an exposed-aggregate finish, care should be taken to prevent the reinforcement being splashed with the liquid. Release agents, too, should not be allowed to come into contact with the bars.

6.3 Fixing reinforcement

Unless the bars are rigidly fixed in the correct position, the reinforcement may be displaced during concreting, particularly if the concrete is vibrated. Top layers of horizontal steel

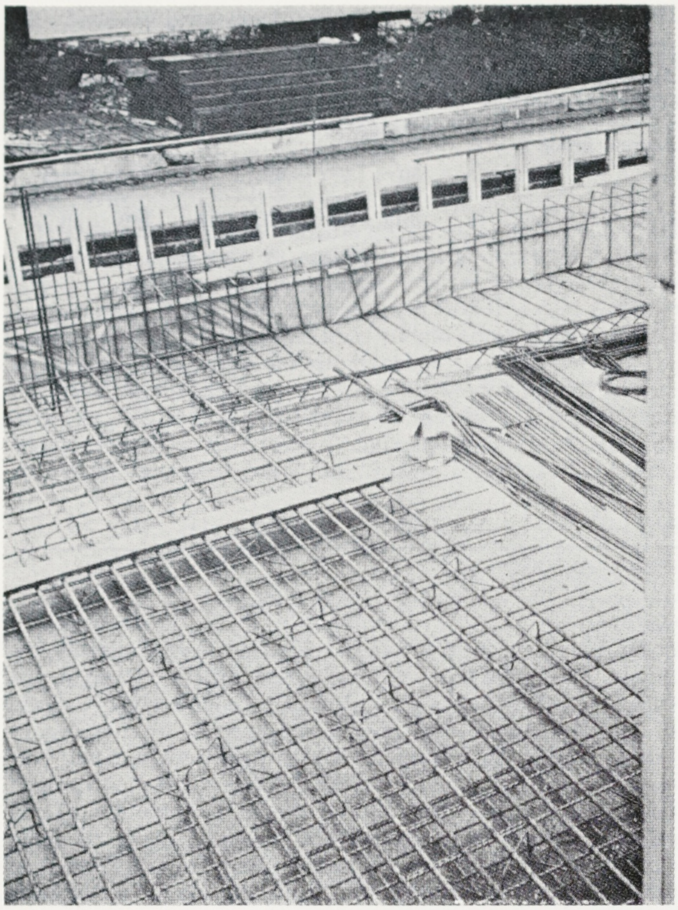

FIG. 4

Reinforcement fixed in place for a reservoir slab. Note the supporting chairs.

should be well supported on steel chairs (Fig. 4) so that they are not displaced by operatives walking on them or by other excessive loads such as bundles of unfixed reinforcement—both of which should be actively discouraged as far as possible. Special care should be taken in fixing the top tension steel in cantilevers.

At all intersections, the bars or links should be securely tied together with 16 swg soft iron wire. The ends of the wire ties must not point towards the face of the concrete, and all ends should be cut off or, preferably, bent inwards so that there is no risk of rust staining the surface of the concrete. As soon as steel fixing has been completed, offcuts of binding wire must be removed from the insides of forms, either by blowing them out with compressed air or using an industrial vacuum machine. Instead of binding wire, proprietary makes of spring clips may be used which initially may be more expensive but obviate some of the difficulties encountered with hand tying.

Where the reinforcement is congested or complicated, spot welding may be specified for assembling it. Structural butt welding and lapped joint welding may be carried out on site, so long as suitable safeguards and techniques are employed and the steel has the required welding properties. Generally, however, all welding should be carried out under controlled conditions in a factory or workshop.

It is important to ensure that all reinforcement is correctly placed and fixed before concreting. If there is any uncertainty about the arrangement of the reinforcement, or any discrepancy between the bar schedules and the drawing, the engineer should be consulted immediately.

For foundation work and floors laid on ground, a concrete blinding layer should first be laid in order to create a clean base on which to assemble and fix reinforcement.

6.4 **Cover to reinforcement**

The nominal concrete cover—the distance from the outside of the reinforcement to the concrete surface—should be as given on the working drawings. The actual concrete cover should nowhere be less than the nominal cover minus 5 mm ($\frac{1}{4}$ in). Small, precast concrete blocks of a quality similar to that of the concrete to be placed, or one of the proprietary plastic spacers, may be used for maintaining the correct cover, as illustrated in Fig. 5. All reinforcement projecting above the formwork should be secured to prevent its being displaced while the concrete is being placed.

The cover recommended for concrete made with aggregates or air-cooled blastfurnace slag is as given in Table 7. For concrete made with lightweight aggregate this cover should be increased by 10 mm ($\frac{3}{8}$ in), except for internal non-corrosive conditions. Where lightweight aggregate concrete having a

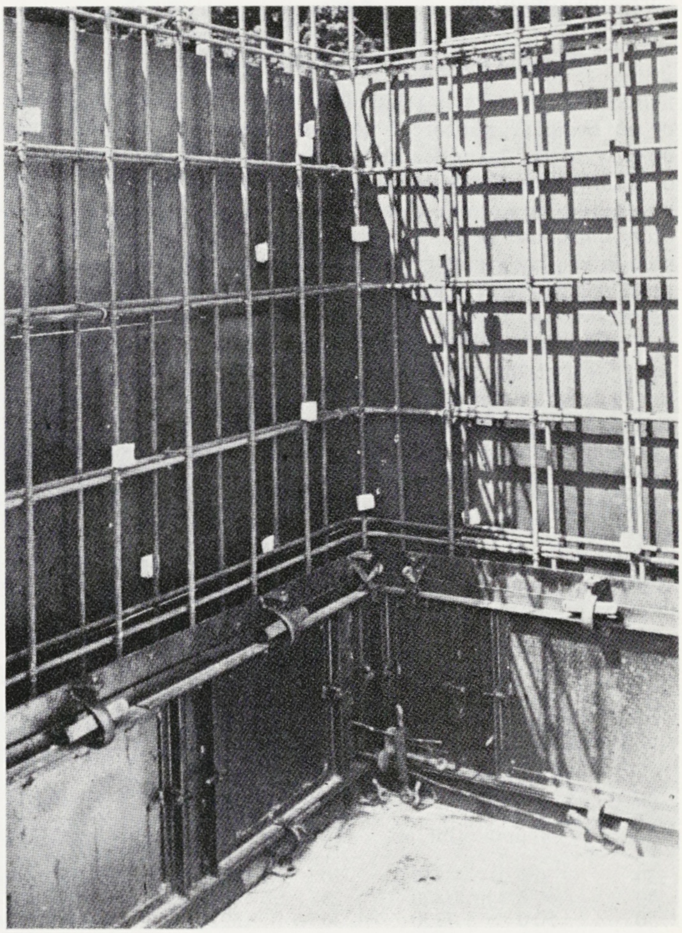

FIG. 5

Mortar bar spacers wired to reinforcement.

TABLE 7

REQUIRED COVER FOR CONCRETE GRADES AND DIFFERENT CONDITIONS OF EXPOSURE

Conditions of exposure	Nominal cover									
	Concrete grade									
	20		25		30		40		50 and over	
	mm	in	mm	in	mm	in	mm	in	mm	in
Mild: e.g. completely protected against weather, or aggressive conditions, except for brief period of exposure to normal weather conditions during construction.	25	1	20	¾	15	⅝	15	⅝	15	⅝
Moderate: e.g. sheltered from severe rain and against freezing whilst saturated with water. Buried concrete and concrete continuously under water.	—	—	40	1½	30	1¼	25	1	20	¾
Severe: e.g. exposed to driving rain, alternate wetting and drying and to freezing whilst wet. Subject to heavy condensation or corrosive fumes.	—	—	50	2	40	1½	30	1¼	25	1
Very severe: e.g. exposed to sea water or moorland water and with abrasion.	—	—	—	—	—	—	60	2½	50	2
Subject to salt used for de-icing.	—	—	—	—	50*	2	40*	1½	25	1

* Only applicable if the concrete has entrained air.

cube strength less than 20 N/mm² (300 lbf/in²) is used, the minimum cover should be 25 mm (1 in) for internal non-corrosive conditions.

7 PRESTRESSED CONCRETE

Because concrete is relatively weak in tension, large areas of an ordinary reinforced concrete member contribute little directly to its flexural strength. For example, the lower part of a simply supported beam is in tension and this tensile stress is ignored in normal calculations. Prestressing makes more economical use of the concrete by enabling all the concrete section to play a part in carrying the load. In prestressed concrete the whole or part of the concrete section is compressed before loading is applied, so that when the concrete is loaded this compressive stress is reduced by flexural tension. The net result is that the concrete never goes into tension, or at least the tensile stresses are kept acceptably low. A prestressed concrete beam is thus like a row of books which can be picked up by two hands pressing horizontally on the ends but which will collapse if the pressure is relaxed.

There are two distinct ways of producing prestressed concrete: by pre-tensioning or by post-tensioning. Pre-tensioned prestressed concrete, which is usually precast, is made by casting the concrete round wires or strands which have previously been stretched between fixed anchorages outside the mould. When the concrete has gained sufficient strength, the wires are released and they attempt to contract, but because of their bond with the concrete the effect is to compress it. Post-tensioned prestressed concrete, which may be either precast or cast-in-situ, uses tendons which are threaded through ducts cast into the concrete. When the concrete has gained sufficient strength the tendons are stressed. This is normally carried out from one end while the other end is anchored in the concrete. When stressed in tension, the ends of the tendons exert a compressive force on the concrete between them. The stress is transferred from the tendons to the concrete by means of proprietary bearing plates and anchorage devices designed to suit the type of tendons and jacking system.

7.1 Concrete for prestressing

Concrete suitable for prestressing must be of a higher strength than is necessary for much reinforced concrete. The minimum characteristic strength for post-tensioned work is 30 N/mm² and that for pre-tensioned work is 40 N/mm². In prestressed concrete design it is necessary to specify the minimum strength at which stressing may be undertaken, and it may also be necessary to know the strength of the concrete on loading. Table 8 gives assumptions about the rate of strength development which may be used for this purpose.

TABLE 8

INCREASE OF STRENGTH OF PRESTRESSED CONCRETE WITH AGE

Grade	Characteristic strength		Cube strength at an age of									
			7 days		2 months		3 months		6 months		1 year	
	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²	N/mm²	lbf/in²
30	30·0	4350	20	2900	33	4780	35	5070	36	5220	37	5360
40	40·0	5800	28	4060	44	6370	45·5	6600	47·5	6880	50	7250
50	50·0	7250	36	5220	54	7830	55·5	8050	57·5	8330	60	8700
60	60·0	8700	45	6520	64*	9280	65·5*	9500	67·5*	9790	70*	10 150

* These increased strengths due to age should be used only if it has been demonstrated to the satisfaction of the engineer that the materials to be used are capable of producing these higher strengths.

7.2 **Steel for prestressed concrete**

Prestressing tendons consist of cold-drawn high-tensile wire, strands helically spun from cold-drawn high-tensile wire, or cold-worked high-tensile alloy steel bars. All must be stored in clean, dry conditions. Prestressing steel must never be welded.

7.2.1 **Wire**

Wire is normally only suitable for pre-tensioned work. Plain hard-drawn steel wire is used in the sizes given in Table 9. Wires of differing tensile strength and relaxation properties are available. The higher strengths are available only with the smaller wires. It is essential that all prestressing wire should be carefully labelled to indicate its category.

Unless the manufacturer is asked for specially wound large-diameter coils, the wire is likely to be supplied in smaller coils which require the wire to be straightened before use. The wire should be inspected to ensure that it has been thoroughly degreased and it must remain uncontaminated by formwork release agents.

It is sometimes thought necessary to use indented or crimped wire in order to increase the ultimate bond strength. This type of wire is available in diameters of 3 mm or larger.

TABLE 9

PREFERRED SIZES OF PRESTRESSING WIRE

Nominal wire diameter	Specified characteristic strength*	
mm	N/mm²	lbf/in²
7	1570	228 000
5	1570	228 000
4	1720	249 000
3	1720	249 000

* The characteristic strength of a prestressing tendon is the ultimate load below which not more than 5% of the test results fall.

TABLE 10

PREFERRED SIZES OF STRAND FOR PRESTRESSING

Type of strand	Diameter		Characteristic load		Nominal cross-sectional area	
	mm	in	kN	lbf	mm²	in²
7-wire	12·5	0·49	165	37 200	94·2	0·146
	15·2	0·60	227	51 100	138·7	0·215
19-wire	18·0	0·71	370	83 300	210	0·325
	25·4	1·00	659	148 400	423	0·655
	28·6	1·125	823	185 200	535	0·828
Compacted	13·0	0·51	230	51 800	120	0·186
	15·2	0·60	300	67 500	165	0·256
	18·0	0·71	380	85 500	223	0·345

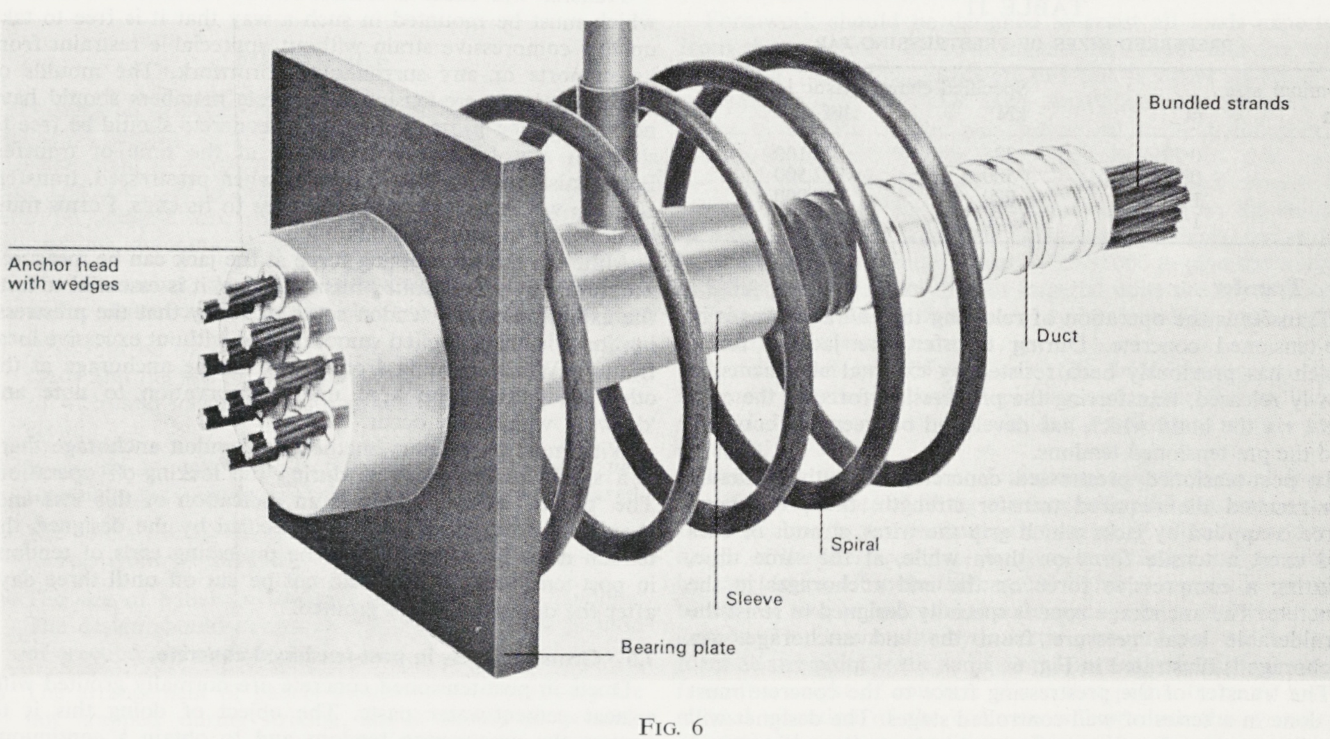

FIG. 6

Wire anchorage.

7.2.2 **Strand**

Strand consists of several high-tensile wires layered together like a rope. There can be one or more layers around a central core wire. The most commonly used strands are made up as follows:—

Single layer	(Seven-wire)	6 over 1
Multi-layer	(Nineteen-wire)	6 over 6
		over 6
		over 1

Compacted strand is strand which has been drawn through a die after being layered. Strand up to 18 mm dia. may be used for pre-tensioned construction. Strand is convenient for post-tensioned work because a high load can be concentrated in one anchorage, and is also useful where wire cannot be used because of the difficulty of straightening the coils as supplied.

Strand makes use of the higher strength capacity of small-diameter cold-drawn steel. It can be tensioned by the single action of one jack, which is not always possible with a number of separate wires of the same total prestressing capacity. Although groups of separate wires can be tensioned together, strand has the advantage that a greater prestressing force can be concentrated in the same anchorage area.

7.2.3 **High-tensile alloy steel bars**

Cold-worked high-tensile alloy steel bars are normally used for straight tendons, most frequently in post-tensioned work. Bars are usually supplied in lengths up to 18 m. The ends of the bars are threaded so that the jack can grip the bar and allow the stressing force to be locked in by special nuts. One advantage of bars is that their threaded ends make it easier to couple on another length of tendon in sectional construction such as cantilever bridge work.

TABLE 11

PREFERRED SIZES OF PRESTRESSING BAR

Nominal size		Specified characteristic load	
mm	in	kN	lbf
20	0·79	325	73 100
25	0·98	500	112 500
32	1·26	800	180 000
40	1·57	1250	281 000

7.3 **Transfer**

Transfer is the operation of releasing the end anchorages in pre-tensioned concrete. During transfer, the jacking force, which has previously been resisted by external abutments, is slowly released, transferring the prestressing force to the concrete via the bond which has developed between the concrete and the pre-tensioned tendons.

In post-tensioned prestressed concrete, when the concrete has reached the required transfer strength, the prestressing force is applied by jacks which grip the wires, strands or bars and exert a tensile force on them while, at the same time, exerting a compressive force on the end anchorages in the concrete. The anchorage zone is specially designed to resist the considerable local pressure from the end anchorage. An anchorage is illustrated in Fig. 6.

The transfer of the prestressing force to the concrete must be done in a series of well-controlled stages. The designer will give instructions for the programme and sequence of tensioning and, where there is more than one anchorage, he will specify the amount of force to be induced in each tendon at each stage. It is common practice to exceed the design prestress for a short time in order to reduce the subsequent relaxation of stress in the steel, but this should only be done strictly in accordance with the designer's instructions.

7.4 **Prestressing operations**

The jacking of prestressing tendons is a potentially dangerous operation. A great deal of energy is contained in a stretched tendon and should the tendon fail, or a slip occur at the jaws of the jack, this energy is violently released. When stressing tendons, either in pre-tensioned construction before the concrete is cast or in post-tensioned construction when the concrete is sufficiently mature, the following procedure should always be adopted.

No person, whether spectator or jack operator, should stand in line with the tendon and its anchorage until after jacking is complete and the jack removed. A heavy steel screen should be provided behind the jack if, for any reason, the area cannot be kept clear. The process of stressing is a proving one for tendons, anchorages and jacks, and serious accidents have been known to occur when these simple precautions were not taken.

The tendons are most frequently stressed by hydraulic jacks, either power- or hand-operated. The tension which has been induced in the tendon is calculated either by measuring the extension of the tendon or by noting the hydraulic pressure registered by the gauge or, more usually, by both methods. Once an initial tension has been applied, the stretching of the tendon should be reasonably proportional to the increasing jack force. In post-tensioned work, an allowance must be made for the loss of prestressing force due to friction and misalignment of the tendons. Careful inspection of the duct alignment for accuracy and smoothness of profile is essential, because permitted deviations are small.

Transfer will cause a contraction in the concrete member, which must be mounted in such a way that it is free to take up this compressive strain without appreciable restraint from its supports or any surrounding formwork. The moulds or forms used for pre-tensioned concrete members should have no interlocking projections and the concrete should be free to slide on any formwork remaining at the time of transfer. Beams also tend to 'hog' upwards when prestressed, transferring the whole weight of the member to its ends. Forms must be designed to withstand this.

Although the prestressing force at the jack can be measured accurately by a hydraulic pressure gauge, it is essential to note the extension of the tendon so as to verify that the prestressing force is being applied smoothly and without excessive local friction. When stressing from one end, the anchorage at the other end should be kept under observation to note any 'draw-in' which may occur.

With most proprietary methods of tendon anchorage there is a small loss in prestress during the locking-off operation. The 'pull-in' of the tendon is an indication of this loss and, should this exceed the amount specified by the designer, the tendon must be re-tensioned. The projecting ends of tendons in post-tensioned work should not be cut off until three days after the ducts have been grouted.

7.5 **Grouting ducts in post-tensioned concrete**

Ducts in post-tensioned concrete are normally grouted with a neat cement-water paste. The object of doing this is to protect the prestressing tendons and to obtain a continuous bond between them and the surrounding concrete. To do this,

the grout must completely fill the ducts. Where no metal sheathing has been used to form a duct, the duct should first be flushed with water to ensure that the concrete is wetted and concrete chippings removed. The water should then be blown out by oil-free compressed air. End anchorages are normally provided with grout injection points and it is usual for further injection points to be provided at the lowest points of curved ducts.Vent holes should be provided where air might otherwise be entrapped, such as at the highest points of undulating tendons. Grout is pumped into the injection points until it flows freely out of the vent hole, which is then sealed. Where there is more than one vent hole, these should be sealed consecutively in the direction of flow. The injection points should be maintained under pressure until the grout has set.

Vertical ducts should be grouted from the bottom. It is beneficial to extend the top of the duct with a tube so that any bleeding or settlement of the solids in the grout takes place outside the concrete. This leaves a column of weak grout above the work, which can be broken off later.

It is important to prevent water separating from the grout because entrapped water in the duct reduces the protection of the steel tendon and is a serious threat to any member exposed to frost. The water/cement ratio of the grout must therefore be kept as low as possible and should normally not exceed 0·45. An expanding admixture may be beneficial, but no admixture containing chlorides or nitrates should be used. It is most important that the admixture should be thoroughly dispersed in the grout and that the grout should be thoroughly mixed for at least two minutes.

The concrete should be protected from any vibration for at least three days after grouting. In winter it may be advisable to protect the concrete from frost for as long as 28 days after grouting, to prevent water freezing in the duct and splitting the member.

8 FORMWORK

The purpose of formwork is to contain freshly placed and compacted concrete until it has gained enough strength to be self-supporting, to produce a concrete member of the required shape and size, and to produce the desired finish to the concrete. To achieve this, the general design and construction requirements of formwork are as follows:

(1) The formwork should be sufficiently rigid to prevent undue deflection during the placing of the concrete.

(2) It should be of sufficient strength to carry the working loads and the weight or pressure of the wet concrete and to withstand incidental loading and vibration of the concrete.

(3) It should be set to line and level within the specified tolerance and include any camber which may be required.

(4) The joints should be sufficiently tight to prevent loss of mortar from the concrete.

(5) The size of panels or units should permit easy handling. The design should permit an orderly and simple method of erection and striking.

(6) The arrangement of panels should be such that they are not 'trapped' during striking, and it should be possible to strike side forms from beams without disturbing the soffit formwork.

8.1 Types of formwork

In recent years the number of types of formwork has grown considerably, although traditional methods using materials such as timber are still very common. Formwork facing materials include timber, plywood, steel, concrete, glass-reinforced plastics, hardboard and expanded polystyrene. In addition, form liners such as rubber, thermoplastics or other sheet materials may be used. Some liners are re-usable but other materials need to be renewed after one use.

Several proprietary formwork systems are available, and large jobs often make use of forms specially designed as a system for that job. Precast concrete, woodwool slabs or other materials may be used as permanent formwork.

The system of falsework supporting the formwork must be designed to withstand the loads imposed on it. Tubular steel scaffolding or adjustable proprietary steel props are the most common forms of support, although timber is still used on some sites and heavy-duty shoring and specially designed supporting systems are often required. When adjustable steel props are used it is most important that they be installed so that they are vertical and loaded axially and that the hardened steel pins provided by the manufacturers are used.

Slipforms are often used for walls, lift shafts and building cores, silos, towers, chimneys, and tunnel and shaft linings. This type of form is moved almost continuously, usually by means of hydraulic jacks, leaving concrete of the required shape and dimensions behind. Slipforming saves time by eliminating the task of striking and re-setting formwork and by allowing continuous concreting, but it is not normally an economic solution for vertical structures less than about 15 m (50 ft) high. The design and operation of slipforms require considerable experience and are usually undertaken only by specialist sub-contractors.

8.2 Design of formwork

Formwork should be designed to resist all loads expected from the concrete, the environment, live loads and the self-weight of the forms, although this last is often so small in comparison with the other loads that it may be neglected.

For formwork design, the density of normal concrete is usually taken as 2400 kg/cm^3 (150 lb/ft^3), so that each 100 mm (4 in) thickness of slab produces a load on a soffit form of about 240 kgf/m^2 (50 lbf/ft^2). Similarly, the weight of a beam can be obtained by multiplying the depth and width in millimetres, then multiplying by 2·4/1000 to give the weight in kilograms per metre run. In imperial units the approximate weight in pounds per foot run may be found by multiplying the depth and width in inches.

A live load of 375 kgf/cm^2 (75 lbf/ft^2) is commonly assumed, but the method of placing and other factors liable to load the form should be considered separately for the particular site.

The horizontal pressure of concrete against vertical forms depends on the rate of filling the form, the thickness of the concrete section, the temperature, the degree of vibration and the height of the lift. Economies in formwork design may be made by considering all these factors, but in many cases it is enough to assume simple hydrostatic conditions with an allowance for impact. The horizontal hydrostatic pressure on the vertical form at any point is the same as the vertical pressure resulting from the weight of concrete above that point. In many cases, normal vibration of the concrete will produce this pressure, whatever the other conditions. Particular care is needed to provide an adequate number of form ties where these are

used to link together the opposite panels of a wall form. Whereas slightly inadequate design of other elements of the formwork may lead to large deflections or leakage, the failure of form ties can more easily cause a dangerous collapse.

The strength of formwork, although very important, is often secondary to its stiffness, which must be sufficient to prevent its deflecting significantly under load, otherwise the resulting concrete surface will show the deformation. When using plywood, it is important to recognize that the stiffness parallel to the face grain is greater than the stiffness at right-angles to this direction. Formwork must be watertight, because small leaks lead to unsightly stains on the concrete surface and large leaks can cause honeycombing. The use of foamed plastic sealing strips is a common and effective means of sealing joints. Joints may also be sealed by adhesive tape.

8.3 **Surface treatment**

Where the appearance of the concrete is of importance, it is vital that care be taken with the surface of the form. All marks on the form, such as vibrating poker 'burns', as well as varying properties in the form-face material, such as uneven water absorbency in timber, will show on the finished concrete. Forms should have all loose wire and other debris cleaned out of them prior to concreting; this is usually done with a compressed air hose. It is particularly important that all steel particles be removed as they will rust and spoil the final appearance of the concrete. There is still need for a good finish even when grit-blasting or bush hammering is to be used to expose the aggregate. It is a fallacy to think that bush hammering will make a poor finish acceptable. In practice, the bush hammering of a poor finish serves to accentuate the surface defects.

To prevent damage to the concrete, the surface of the form must be coated with a release agent prior to concreting. There are various types of release agent, the most useful are probably neat oils with surfactants, mould cream emulsions and chemical release agents. Release agents should be applied to give a very thin film. A common fault is the use of too much oil. If the oil is thin, application by an airless spray is recommended. Thicker oils may be applied by brush or cloth and spread as far as possible, all excess being removed with a cloth. Waxes may be applied with a squeegee. The use of barrier paints produces a hard wearing surface and may extend the life of timber or plywood forms. If paint is not used, three coats of mould oil should be applied before the form is used for the first time. Some barrier paints are not suitable for use with certain tropical hardwoods, and the manufacturer's advice should be sought on this point. To avoid contamination of the reinforcement, the release agent should be applied before the forms are erected, but to do this it may be necessary to protect the forms from the weather.

Unpainted timber forms become progressively less absorbent as the pores of the wood become filled with cement paste during use. This affects the appearance of the finished work—an abrupt colour change will be seen on the concrete—so new and old materials should not be used alongside each other. Similarly, patches of new material in old formwork will produce noticeable colour changes.

In a similar way, painted timber and various types of plastics forms having a glazed or glossy surface produce an appearance which changes with the number of uses. In this case the surface glaze is reduced by the first use and subsequent uses produce a less highly polished surface on the concrete. Some plastics-faced plywoods give a similar effect. The first few uses of these materials can sometimes produce a very hard, dense and almost black surface to the concrete. This is probably caused by slight movement of the form at a critical time during the hardening process. Normally it occurs only with impervious form faces. Similar discolouration of the surface of concrete placed against a steel form can usually be attributed to the presence of mill scale on the steel.

8.4 **Striking of formwork**

The period which should elapse before the formwork is struck will vary from job to job and will depend on the concrete used, the weather and exposure of the site, any subsequent treatment to be given to the concrete, the method of curing and other factors. Formwork must not be removed until the concrete is strong enough to be self-supporting and able to carry imposed loads. Thus the time of striking should be related to the strength of the concrete and, obviously, soffit forms to beams and slabs must be left in place longer than is necessary for the side forms.

In general, soffit forms should not be struck until the concrete has achieved a cube strength, measured on cubes cured damp and kept next to the site concrete, of at least twice the stress to which the concrete will be subjected at the time of striking.

TABLE 12

MINIMUM PERIODS BEFORE STRIKING FORMWORK

Formwork	Surface temperature of concrete 16°C	7°C
Vertical formwork to columns, walls and large beams	9 hours	12 hours
Slab soffits (props left under)	4 days	7 days
Beam soffits (props left under)	8 days	14 days
Props to slabs	11 days	14 days
Props to beams	15 days	21 days

Subject to the requirements of the specification, and where no other information is available, the periods given in Table 12 may be taken as a general guide for the removal of the formwork. The periods apply to ordinary Portland cement concrete. When rapid-hardening Portland cement is used these periods may be reduced, but it is preferable to make test cubes rather than to rely on an abitrary reduction.

In cold weather, the times given in Table 12 should be increased to allow for the reduced maturity. For example, it would be appropriate to add half a day to the times for 7°C for each day on which the temperature is generally below 7°C (45°F) and a whole day for each day on which the temperature is below 2°C (35°F). In winter, when the average temperature is about 5°C (41°F), it would usually be necessary to double the times shown. This is discussed in greater detail in Section 5.7.

Striking must be carried out with care to avoid damage to arrises and projections, and it may be necessary to protect some of the work from damage immediately after removing the forms. Before soffit forms and props are removed, the concrete surface should be exposed carefully, in order to ascertain that the concrete has hardened sufficiently.

Curing should start immediately after the removal of formwork and, if necessary, the concrete should be insulated as a protection against low temperature. Timber formwork is frequently a good insulator in its own right, so in winter it

is particularly important to avoid thermal shock to the warm concrete when timber or insulated steel forms are removed and the concrete is exposed to the air. If the formwork is not required elsewhere, it may suffice to leave it in place until the concrete has cooled from its high early temperature. The formwork must be removed slowly: the sudden removal of wedges is equivalent to a shock load on the partly hardened concrete. Careful removal is also less likely to damage the formwork itself and will thus prolong its life.

8.5 Care of formwork

The formwork for concrete frequently accounts for over a third of the cost of the finished concrete, so it should be handled with care. The life of forms can be extended considerably by careful treatment, thus decreasing the overall cost of the job. Rough treatment may make timber and plywood forms useless after one pour, whereas eight or more uses may be obtained by following good site practice.

Formwork should be stored under cover and off the ground. Plywood will tend to fail by delaminating, so edge protection is important. Oiling should always be extended round the edges and, even if paint is not used on the face of the form, two or three coats of barrier paint on the edges will help to preserve them. Steel forms should be prevented from rusting, or rust stains will be transferred to the finished concrete. To avoid damage during lifting, large forms should be handled by a lifting beam or suspension from the lifting points specified by the designer. Formwork should be kept free of dirt, particularly rust from reinforcement, as this can produce unsightly staining on the concrete.

9 CURING

9.1 Purpose of curing

The setting and hardening of cement depend on the presence of water. Drying out, if allowed to take place too soon, results in low strength and a porous concrete. At the time the concrete is placed, there is normally an adequate quantity of water present for full hydration, but it is necessary to ensure that this water is retained so that the chemical reaction continues until the concrete has thoroughly hardened. Methods of curing should be designed to maintain the concrete above freezing point and in a continuously moist condition for several days, either by preventing evaporation or by keeping the surface of the concrete continuously wet.

The lower the temperature, the slower is the rate at which concrete hardens. If the temperature of the plastic concrete falls below freezing point, the freezing and resulting expansion of the water cause permanent damage.

If curing is efficient, the strength of the concrete increases with age; this increase is rapid at early ages and then continues more slowly for an indefinite period. Correct curing increases the impermeability and durability of the concrete, which is particularly important when it will be subject to water pressure or severe environmental conditions.

Curing increases resistance to abrasion; effective curing is thus most necessary for floors and other surfaces subject to wear. Continuous curing from the time the concrete is placed helps to ensure a hard, dense surface and to reduce the risk of crazing and dusting.

A further reason for curing is to reduce evaporation of water from the fresh concrete in hot weather because unless this is checked, 'plastic' or 'wind' cracks may appear while the concrete is still plastic. This has been discussed in Section 4.

9.2 Methods of curing

To prevent evaporation of moisture and the consequent formation of cracks in the surface, the curing of horizontal surfaces exposed to the sun or to drying winds must begin immediately the concrete has been placed and finished. One method is to spray the surface with a composition which forms an impervious membrane preventing evaporation. Some of these curing membranes contain a dye which may discolour the finished surface. Other membranes contain aluminium powder to reflect much of the heat from the sun which might otherwise raise the temperature of the concrete excessively. Membranes are not suitable for floors where further coatings such as screeds have to be bonded to the surface, nor are they suitable indoors because they are designed to flake off and disintegrate under the action of sunlight.

As an alternative to sprayed membranes, tents may be erected over the concrete to shade it from the sun. In paving work the tenting may be carried on frames spanning the work and running on wheels on the side forms so that it can be moved forward as the work progresses. It is most important that the sides of the tenting should extend to ground level to prevent the draughts which cause plastic cracking. There is also a danger that tenting may funnel the wind down its length, thus defeating its object. A further alternative is to use thin plastic sheeting or waterproof paper to cover the freshly placed concrete. These, however, tend to mark the surface or to blow away if not weighted down.

At seasons of the year when the sun is not strong and there are no severe drying winds, curing may be postponed until the concrete has set. In this case, plastic sheeting or waterproof paper may be used and weighted down, care being taken not to damage the surface. Alternatives are matting, fabric, or clean sand kept damp for at least seven days. Where tenting is used until the concrete has set, it should be replaced as soon as possible with a waterproof or damp covering, as described above. It is sometimes possible to flood horizontal slabs with water after they have set, but this should not be done while the concrete is still plastic or if there is a risk of frost.

The curing of vertical surfaces is more difficult. It may be done initially by leaving the forms in place, or by covering with a plastic sheet. Membranes can sometimes be used, but are not generally suitable where any subsequent treatment or rendering is to be used. Water sprays, or damp canvas or hessian, should be used only when the wall has cooled down to ambient temperature, otherwise any tendency to crack due to heat of hydration movement may be worsened.

Plastic sheeting or waterproof paper is recommended for curing white or coloured concrete surfaces.

Arrangements should be made for a particular person on the contractor's staff to be responsible for curing. The man should be briefed on the importance and method of curing to be used on the job. Adequate curing facilities should be made available on site before any concreting starts.

Concrete should be protected from damage by construction traffic or loading until at least the end of the curing period and until the concrete has adequate strength to withstand the loading.

10 TESTING OF CONCRETE AND CONCRETING MATERIALS

Most tests on concrete and concreting materials fall into one of the following categories: —

(1) Tests to assist in deciding whether particular sources of material are suitable for concrete and, if so, in what proportions they should be combined. These tests are usually carried out by experienced staff in a laboratory. They may be omitted for many routine concreting jobs where the materials are known to be of good quality, e.g. where they are known to comply with relevant Standards. They should not be omitted if special properties such as very high strength are required.

(2) Site tests to supplement laboratory tests and to ensure that the quality of the concrete is both acceptable and reasonably uniform throughout the job. All the tests used on site can be, and often are, carried out in the laboratory. Site tests are reasonably simple and do not require specially skilled staff.

This section concentrates on the site tests, though a few references are made to laboratory tests which on occasion may be used in the field.

10.1 Sampling of materials

The object in sampling is to produce a manageable quantity of material which is truly representative of the consignment being sampled. Aggregate and concrete are heterogeneous materials and the storage of aggregate can present problems of access, so great care is needed in sampling if reliable test results are to be obtained. Materials being delivered to site in relatively small lots are best sampled during delivery to stockpiles or silos.

10.1.1 Cement

The requirements for sampling cement are given in Section 2.1.4. As noted there, cement testing is rarely required on site.

10.1.2 Aggregates

When sampling under favourable conditions, such as exist when the materials do not vary greatly from point to point in the mass, a main sample should be made up from at least twelve small portions or increments taken from different places in the stockpile. The main sample of each type or size of aggregate to be tested should contain at least the quantities given in Table 13. This main sample may subsequently be reduced to a smaller quantity, where appropriate.

Sampling is best carried out when aggregate is being loaded or unloaded from a vehicle, or when it is being discharged from a conveyor belt. In each case, the increments should be taken at fairly regular intervals during the movement of the quantity being sampled. In other conditions, a greater number of increments will be required so as to obtain a larger sample than the minimum given in Table 13. One situation which presents difficulties in sampling is when vehicles have been 'sandwich - loaded' with different sizes dispensed one after another from overhead bins. This is a most undesirable practice and in such cases the sampler should use his experience and discretion to ensure that a representative sample is obtained.

The reduction in size of a sample must be carried out with care to ensure that the smaller sample is representative of the larger. A riffle box may be used when the aggregate is surface-dry. A riffle box is designed so that material poured in the top is split approximately equally and diverted to two sides; material to one side of the box is discarded and the remainder tested or split to a smaller sample.

The method known as quartering is more frequently used. The aggregate should be damp and thoroughly mixed and piled into a heap on a clean, hard surface. The heap is then flattened to an even layer 75 to 100 mm (3 to 4 in) thick. It is then divided into equal quadrants and the two diagonally opposite quarters are discarded. The remaining quarters are well mixed together and the process is repeated until the desired size of sample remains.

TABLE 13

MINIMUM WEIGHTS FOR SAMPLING AGGREGATES

Nominal size of aggregate	Minimum weight of sample despatched for testing kg	lb
25 mm (1 in) and larger	50	112
Smaller than 25 mm (1 in) but larger than 5 mm ($\frac{3}{16}$ in)	25	56
5 mm ($\frac{3}{16}$ in) or smaller	13	28

When samples of aggregate are to be sent to a laboratory, they should be packed securely in containers which will prevent loss of the fine dust and damage in transit. Heavy-duty polythene bags are often used.

10.1.3 Concrete

Correct sampling of concrete is essential if the test results are to be representative of the bulk of the concrete.

Whenever possible, the sampling should be done when the concrete is moving in a stream, such as when it flows down the discharge chute of the mixer or is being conveyed on a belt. Concrete may be sampled from a stationary lorry or heap, but this method is less satisfactory. Concrete cannot be sampled satisfactorily from a discharging lorry or dumper.

Samples taken from a falling stream, conveyor or chute should consist of not less than four increments, and the person taking the sample should be able to reach the whole cross-section of the stream; biased samples may result if part of the stream is difficult to reach. Increments should be taken at equally spaced intervals; thus, when four increments are required to make up the required test sample, the increments should be taken about the time when one-fifth, two-fifths and four-fifths of the concrete have been discharged.

When sampling from lorries or heaps, some bias will be inevitable as not all the material is freely accessible. The sample should consist of not less than six increments which should, wherever possible, be distributed through the depth of concrete as well as over the exposed surface.

It is preferable to use a standard-size scoop when sampling. This should hold an increment weighing about 5 kg (11 lb) when the operator is sampling from a moving stream and 3·5 kg (8 lb) when he is sampling from a heap.

Where the point of mixing and the point of placing are some distance apart, there is the choice of taking samples at either place. Sampling and testing at the mixer has the advantage of enabling adjustments to the mix to be made more quickly, but workability tests can be more easily related to the placing conditions if done at the point of placing. On some jobs it may be useful to carry out a few tests at both places so as to ascertain, for example, the change in workability during transport in hot weather.

10.2 Testing of materials

10.2.1 Cement

The testing of cement must be carried out, according to the relevant Standard, under very carefully controlled conditions; for this reason it is rarely possible to carry out the tests on site. For example, when specimens are being made for strength

tests the air of the mixing room must be kept between 18°C and 23°C, the air in the immediate vicinity of the specimens during the first 24 hours of storage must be at a temperature of 19 ± 1°C and at least 90 per cent relative humidity, and the temperature of the curing water must be 19 ± 1°C. The British Standard tests specified for cements[1-6,32] cover fine ness, chemical composition, strength, setting time, soundness and, in special cases, heat of hydration.

10.2.2 Aggregates

In controlling the quality of aggregates, it is important to ensure that the aggregate is clean and does not contain any organic impurities which might retard or prevent the setting of the cement, and that the proportions of the different sizes of particles within a graded material remain uniform.

10.2.2.1 Cleanness

Accurate tests for determining the proportion of clay, silt and dust in fine or coarse aggregates are suitable only for the laboratory. On site, cleanness can be assessed visually, though for natural sands the 'field settling test' will give an approximate guide to the amount of clay or silt.

The test entails placing about 50 ml (millilitres) of a 1 per cent solution of common salt in water (roughly one slightly heaped teaspoonful per pint) in a 250 ml measuring cylinder. Sand, as received, is then added gradually until the level of the top of the sands is at the 100 ml mark and more solution is added to bring the liquid level to the 150 ml mark. The cylinder is shaken vigorously, and the contents allowed to settle for three hours. The thickness of the silt layer which settles above the sand is then measured and expressed as a percentage of the height of the sand below the layer. The amount of clay and silt in the sand may be considered acceptable if it does not exceed 8 per cent.

If a measuring cylinder is not available, a jam jar or bottle filled to a depth of 50 mm (2 in) with sand and to a total depth of 75 mm (3 in) with the salt solution will give comparable results if the contents are allowed to settle for three hours. The thickness of the silt layer in this case should not be more than 3 mm (⅛ in).

The field settling test gives only an approximate guide. Sands apparently containing more than 8 per cent of clay or fine silt cannot be regarded as having failed to comply with the specification, and further laboratory tests to assess their suitability must be carried out.

There is no suitable site test for the cleanness of coarse or all-in aggregates or of crushed-rock fines, and reliance is usually placed upon the grading analysis (*see* Section 10.2.2.3) to show whether there is an excess of fine dust in the material. Problems arise, however, because of the tendency for fine dust to adhere to the rough surface of crushed coarse aggregate.

Similarly, there is no suitable test for the cleanness of a gravel coarse aggregate. It is important to ensure that aggregate particles are not coated with clay and that lumps of clay are not mixed in with the aggregate. The presence of clay indicates that the aggregate has not been washed adequately before delivery or that the aggregate has subsequently become contaminated.

10.2.2.2 Organic impurities

Coarse aggregates from any source, and crushed-rock fine aggregates, are unlikely to contain organic impurities, though natural sands may do so. A laboratory method of test for fine aggregate is based on the measurement of the pH values of aggregate-cement mortars under standard conditions.

10.2.2.3 Sieve analysis

The grading of an aggregate is found by passing a representative sample of dry aggregate through a series of sieves, starting with the largest mesh. If the sieving is carried out by hand, each sieve is shaken separately over a clean tray for not less than two minutes. For many routine purposes mechanical sieving is advantageous, but if this method is used care should be taken to ensure that sieving is complete.

The material retained on each sieve, together with any material cleaned from the mesh, is weighed or recorded. The amount passing each sieve is then calculated as a percentage by weight of the total. Table 14 gives an example of a method for recording a sieve analysis and calculating the percentage passing each sieve.

TABLE 14

EXAMPLE OF THE METHOD OF RECORDING SIEVE ANALYSIS OF AGGREGATE

Sieve size		Weight retained on each sieve g	Total weight passing each sieve g	Percentage passing each sieve
10 mm	⅜ in	0	1348	100
5 mm	$\frac{3}{16}$ in	95	1253	93
2·36 mm	No. 7	160	1093	81
1·18 mm	No. 14	135	958	71
600 μm	No. 25	361	597	44
300 μm	No. 25	342	255	19
150 μm	No. 100	162	93	7
Sieve pan		93		
TOTAL		1348		

Sieving will not be accurate if there is too much material left on any mesh after shaking.

The size of the sample tested depends upon the maximum size of the aggregate. For nominal 40 mm (1½ in) aggregate the sample should weigh at least 15 kg (33 lb), for 20 mm (¾ in) at least 2 kg (5 lb), for 10 mm (⅜ in) at least 0·5 kg (1 lb), and for fine aggregate at least 0·2 kg (½ lb).

The results are plotted on a chart so that the specified gradings and the sample gradings can be more easily compared. It should be noted that the points representing the percentage of material passing the various sieve sizes are joined by straight lines and not by curves.

10.2.2.4 Moisture content

The purpose of measuring the moisture content of aggregate is to enable an estimate to be made of the quantity of water contained within it so that the water added at the mixer gives the required total in the mix. Changes in the moisture content of the aggregate do not justify making adjustments to the weight of aggregate in each batch of concrete, although the batch weight of dry aggregate should be adjusted at the beginning of a job, and during the job if necessary, to allow for an average moisture content. There are various methods whereby the moisture content may be determined.

(i) *Drying methods.* Methods involving the drying of representative samples of aggregate are often used. Site staff usually use the 'frying pan' technique in which the aggregate is dried by heating it in an open pan. The aggregate is first weighed

(W_1), then dried and reweighed (W_2). The moisture content is then calculated as:

$$\frac{W_1 - W_2}{W_2} \times 100\%$$

The sample of aggregate to be tested should weigh between 1·8 and 2·2 kg (4 and 5 lb) for coarse aggregate. The sample size for fine aggregates may be reduced to not less than 0·5 kg (1 lb) if an adequate balance is used for weighing.

This method gives the water content as a percentage of the 'saturated surface-dry' (SSD) weight of the aggregate. This is the measure of moisture content generally used on site; it takes into account water on the surface of all particles of aggregate, but does not include water absorbed into the pores of the aggregate. As an alternative to the saturated surface-dry moisture content, the 'total' moisture content, which includes water absorbed into the pores of the aggregate particles, is sometimes used. When the total moisture content is to be measured, the aggregate should be dried thoroughly: it is necessary to heat the aggregate at 105 $\pm$ 5°C in an oven overnight. An important point to note is that a saturated surface-dry moisture content must be used in conjunction with a specified saturated surface-dry water/cement ratio, often known as the 'free' water/cement ratio: similarly, a total moisture content must be used with a 'total' water/cement ratio. When the SSD moisture content is measured, coarse aggregate should be dried until surface moisture has evaporated (this is often accompanied by a slight change in colour) but any further heating should be avoided. Fine aggregate should be dried until it just fails to adhere to a glass rod when stirred.

If an open source of heat is used it is important not to overheat the aggregate or to heat it too rapidly, which could cause the particles to break up and spit out of the pan. For site work, it is satisfactory to heat the aggregate slowly and to re-weigh it immediately.

(ii) *Displacement methods.* The moisture content of an aggregate can also be determined by one of the displacement methods such as the siphon can, steelyard or measuring cylinder. For all these methods of test, it is necessary to 'calibrate' the particular size and type of aggregate for which the moisture content is required before the moisture content itself can be determined. This is because the moisture content is determined from the difference in weight between equal volumes of dry and wet aggregate or, conversely, the difference in volume between equal weights of dry and wet aggregate.

The advantage of a displacement method is that, once the calibration for a particular aggregate is known, the moisture content can be determined within a few minutes. It is therefore possible to make allowance for the moisture in the aggregate before concreting starts. Samples can even be taken from the batched aggregate and tested before the final adjustment of the water content of the mixed concrete is made.

(iii) *Other methods.* There are other methods of determining the moisture content of an aggregate. Most are proprietary methods and instructions for carrying out the test are supplied with the apparatus.

In the calcium carbide method, using the 'Speedy' apparatus, a sample of fine aggregate is mixed with an excess of calcium carbide in a sealed metal flask. The pressure produced by the acetylene liberated by the reaction between water and carbide is related to the moisture content, which can be read off a dial. Like the displacement methods this test is very quick, but the size of sample is small and two or three tests may be advisable to give a reliable measure of the moisture content in a stockpile of aggregate.

10.2.2.5 **Bulk density of aggregate**

Laboratory methods of measuring the bulk density of aggregates are not generally suitable for use in mix design; for this purpose, an appropriate practical test should be employed. Despite this statement, the two tests for compacted bulk density (rodded) and uncompacted bulk density (loose) are often used. The appropriate value of bulk density for use on the majority of construction sites is likely to lie between these two alternatives, though in the absence of practical site tests it is recommended that the loose bulk density should be adopted in mix design calculations.

The apparatus includes a cylindrical metal container of approximately 3, 7·5, 15 or 30 litres (1/10, $\frac{1}{4}$, $\frac{1}{2}$ or 1 ft^3) capacity. These containers are used for tests on aggregates of nominal maximum size 6·3 mm ($\frac{1}{4}$ in), 14 mm ($\frac{1}{2}$ in), 25 mm (1 in) and 50 mm (2 in) respectively.

The volume of the metal container is first obtained by determining the weight of water at 20° $\pm$ 2°C required to fill it so that no meniscus is present above the rim. The container should then be filled to overflowing with aggregate using a shovel or scoop, the aggregate being discharged from a height of not more than 50 mm (2 in) above the top of the container. Care should be taken to keep segregation of different sizes to the minimum. The surface should be levelled with a straightedge. The aggregates in the measure should then be weighed and divided by the volume of the container to give the bulk density in kg/m^3 (or lb/ft^3) to the nearest whole number.

10.2.2.6 **Mechanical properties of aggregates**

The facilities of a laboratory are needed for determining the mechanical properties of aggregates. The tests include crushing or abrading samples of aggregate to give a measure of its strength or resistance to wear.

10.2.3 **Water**

The most common method is to compare the properties of concrete made with any particular sample of water with those of an otherwise similar concrete made with distilled water.

The tests are, in effect, the same as those for initial setting time and compressive strength for the type of cement being used and will usually be performed in a laboratory.

10.3 Tests for the workability and air content of fresh concrete

The measurement of the workability of fresh concrete is of importance in assessing the practicability of compacting the mix and also in maintaining consistency throughout the job. In addition, workability tests are often used as an indirect check on the water content and, therefore, on the water/cement ratio of the concrete. In this instance, the relationship between water/cement ratio and workability is established in the laboratory or early in the site work; then, by maintaining the correct proportions of cement and aggregate at a constant workability, the water/cement ratio is controlled. Periodic measurements of aggregate moisture content are made to check that the mix proportions are correct.

10.3.1 Slump test

For most concretes, the slump test is a practical means of measuring the workability. Changes in the value of slump obtained during a job may indicate changes in materials in

the water content or in the proportions of the mix, so it is useful in controlling the quality of the concrete produced.

The apparatus consists of a truncated conical mould 100 mm (4 in) in diameter at the top, 200 mm (8 in) at the bottom and 300 mm (12 in) high, with a steel tamping rod 16 mm ($\frac{5}{8}$ in) diameter and 600 mm (2 ft) long, rounded at one end. The inside of the mould should be cleaned before each test and the mould placed on a hard, flat, impervious surface. The mould should be filled in four layers of concrete of approximately equal depth. Each layer is rodded with 25 strokes of the rounded end of the tamping rod; after the top layer has been rodded, the surface of the concrete is struck off level with the top of the mould with a trowel or the rod. Any spillage is cleaned away from around the base of the mould, and the mould is then lifted vertically from the concrete. The slump is the difference between the height of the concrete before and the greatest height after the removal of the mould. If any specimen collapses or shears off laterally, the test should be repeated with another sample of the same concrete; if, in the repeat test, the specimen should again shear, the slump should be recorded together with the fact that the specimen sheared.

If, after the slump measurement has been completed, the side of the concrete is tapped with a rod, a well-proportioned cohesive mix will gradually slump further but a harsh uncohesive mix is likely to collapse.

10.3.2 **Compacting factor test**

The compacting factor test is a more sensitive method of measuring the workability of concrete than the slump test, and enables values to be obtained for mixes which are too dry to give a measurable slump. In this test the density of the concrete after a standardized degree of partial compaction is compared with the density of the concrete after full compaction.

The apparatus consists of two conical hoppers and a cylindrical container mounted vertically above one another. In the test, the top hopper is filled with a sample of concrete which is not compacted. A hinged door at the bottom is released and the concrete is allowed to fall into the lower hopper; then the concrete is similiarly released from the lower hopper and falls into the cylindrical container. The use of two hoppers helps to reduce any inconsistencies caused by uneven loading of the top hopper. Cohesive mixes have a tendency to stick in one or both of the hoppers; if this happens, the concrete may be helped through by pushing a rod gently into the concrete from the top. The concrete above the level of the rim of the cylindrical container is cut off by simultaneously working two steel floats from the outside to the centre.

The weight of concrete in the cylinder is found and is referred to as the 'partially compacted weight'. The corresponding 'weight of fully compacted concrete' is then found by refilling the cylinder in layers approximately 50 mm (2 in) deep, each layer being compacted by hand-ramming or vibration, and trowelling off the concrete level with the rim. The concrete in the cylinder is again weighed, and the compacting factor is the ratio of the partially compacted weight to the fully compacted weight; a higher value indicates greater workability. To ensure a representative result for the workability of a particularly important batch, such as a trial mix, it is advisable to take the average of the values obtained from three tests.

10.3.3 **Vebe consistometer test**

The Vebe consistometer is a satisfactory method of test for measuring the workability of concretes which are so stiff that compaction by vibration is always necessary. It was first developed in Sweden and is accepted in this country for use in the laboratory, but the test is equally useful in the field provided that a suitable three-phase power supply is available.

The test consists of two parts. Firstly, a slump test is carried out inside a hollow cylinder on the top of a vibrating table which, at this stage, is not switched on. The slump is usually very small or zero. Secondly, a transparent plate, which just fits inside the cylinder and which can drop vertically under its own weight, is placed gently on top of the slumped concrete. The vibrating table is then switched on and, under the action of the vibration, its own weight and the weight of the transparent plate, the concrete remoulds itself into the shape of the cylinder. As soon as the concrete is completely remoulded, as can be seen through the transparent plate, the power is switched off. The time taken in seconds for the concrete to be remoulded, known as Vebe degrees, is a measure of the workability of the concrete: a longer time indicates lower workability.

An approximate guide to compacting factors, slump and Vebe degrees suitable for concrete for different purposes is given in Table 15. It must be stressed that the workability tests described above do not measure any fundamental concrete properties and are all arbitrary. Thus there is no absolute correlation between the different methods of test, and the values quoted in Table 15 are approximate only.

TABLE 15

WORKABILITY FOR DIFFERENT PURPOSES

Purposes	Compacting factor	Slump mm (in)	Vebe degrees
Very high-strength concrete for prestressed concrete sections compacted by heavy vibration	0·70–0·78	0	over 20
High-strength concrete sections, paving and mass concrete compacted by vibration	0·78–0·85	0–25 (0–1)	7–20
Normally reinforced concrete sections compacted by vibration. Hand-compacted mass concrete	0·85–0·92	25–50 (1–2)	3–7
Heavily reinforced concrete sections compacted by vibration. Hand-compacted concrete in normally reinforced slabs, beams, columns and walls	0·92–0·95	50–100 (2–4)	1–3
Heavily reinforced concrete sections compacted without vibration. Work where compaction is particularly difficult. Cast-in-situ piling	over 0·95	100–150 (4–6)	0–1

10.3.4 **Air content test**

If an air-entrained concrete is used, the air content of the fresh concrete should be determined. Basically, the test involves measuring the reduction in volume of a known quantity of concrete resulting from an increase in the applied air pressure. The air meter should be of a type in which the air content is read off while the concrete is under an operating

pressure of approximately 1 atmosphere or 0·1 N/mm² (15 lbf/in²). The container of the meter should have a nominal capacity of 0·006 m³, i.e. 6 litres (0·2 ft³).

The container of the meter should be filled with concrete in three approximately equal layers, each layer being compacted with at least 25 strokes of a steel tamping rod 16 mm ($\frac{5}{8}$ in) diameter and 600 mm (2 ft) long (as used for the slump test) or by vibration. The object of ramming or vibrating the concrete is to attain full compaction without removing an appreciable proportion of any deliberately entrained air.

The concrete should be worked until mortar just makes good contact with all sides of the container; prolonged working should be avoided. Provided that the air has been correctly entrained, the amount of air removed by excessive compaction is not likely to be large.

An aggregate correction factor is necessary and will vary with different aggregates. This can be determined only by test, since it is not directly related to the water absorption of the particles. Ordinarily the factor will remain reasonably constant for a particular aggregate, but an occasional check test is recommended.

10.4 Testing of hardened concrete

The 'strength' of hardened concrete is usually measured on specimens which are tested in compression, but sometimes, as in paving work, flexural strength is used. Other tests are available, such as the cylinder-splitting test which gives the indirect tensile strength of the concrete, or non-destructive tests such as the gamma-ray test for the location of voids within the hardened concrete.

10.4.1 Manufacture of test specimens

10.4.1.1 Compressive strength test cubes

Compressive strength tests for concrete with a maximum size of aggregate of up to 40 mm ($1\frac{1}{2}$ in) are usually conducted on 150 mm (6 in) cubes. For aggregate with a nominal maximum size of 25 mm (1 in) or less, 100 mm (4 in) cubes may be used.

The following is a summary of the procedure on site. It should be emphasized, however, that cubes should always be made by men trained in the work and that it is preferable that the same men should make all the cubes throughout the job.

The moulds for test cubes should be made of steel or cast iron with the inner surfaces parallel to each other and machine-faced. Timber moulds cannot be made sufficiently accurately and should not be used. Each mould should have a metal base-plate with a true surface to support the mould and prevent leakage. It is essential to keep the mould and base-plate clean and both should be oiled lightly to prevent the concrete sticking to the sides. No undue force should be used during assembly.

It is essential that the concrete in the cubes should be fully compacted. To assist in ensuring this, a 150 mm (6 in) cube mould should be filled in three layers and a 100 mm (4 in) mould in two layers. When compaction is by hand, each layer should be rammed with *at least* 35 strokes for 150 mm (6 in) cubes and *at least* 25 strokes for 100 mm (4 in) cubes with a steel bar 380 mm (15 in) long, weighing 1·8 kg (4 lb) and having a ramming face 25 mm (1 in) square. Full compaction should be ensured regardless of the number of strokes needed. The ramming of the concrete should be carried out methodicaly, the stroke being evenly distributed over the surface of the concrete in a regular pattern and not concentrated in one particular spot. Alternatively, the concrete may be compacted by vibration, again in layers, using either an electric or a pneumatic hammer or a suitable table vibrator. The surface of the concrete should then be trowelled as smooth as practicable, level with the top of the mould.

Test specimens made on site should be kept at a temperature of 20 ± 5°C in a place free from vibration under damp matting or other suitable damp material, which in turn should be completely covered with plastics or other similar impervious sheeting. They should be kept thus for 16 to 24 hours from the time water is added to the other materials.

After storing, the specimens must be marked for later identification, removed from the moulds (care being taken to ensure the arrises are not broken) and, unless required for test within 24 hours, immediately submerged in a tank of clean water maintained at a temperature of 20 ± 2°C until they are transported to a testing laboratory. While the specimens remain on site, records should be kept of the daily maximum and minimum air and water storage temperatures.

When they are not less than three days nor more than seven days old, the specimens should be packed in damp sand or wet sacks and enclosed, when necessary, in a plastics bag or similar sealed container, and sent to the testing laboratory. They should arrive there in a damp condition not less than 24 hours before the time of test. On arrival at the testing laboratory the specimens should be stored in water maintained at a temperature of 20 ± 1°C until the time of test.

10.4.1.2 Flexural strength test beams

The standard size of beam is 150×150×750 mm (6×6×30 in), but beams of 100×100×500 mm (4×4×20 in) may be used if the nominal maximum aggregate size does not exceed 25 mm (1 in). Moulds are filled and concrete compacted in the same manner as for cubes, by hand or by vibration, but when compacting is done by hand at least 175 strokes of the tamper per layer are needed for 150 mm beams and at least 100 strokes for 100 mm beams. After the top layer has been compacted, the surface of the concrete must be trowelled off level with the top of the mould.

Beams should be cured in the same way as cubes, but it is even more important that the beams should be carefully cured and not allowed to dry at any time before they are tested.

The compressive strength of the concrete can often be measured by using the broken parts of a beam tested in flexure.

10.4.2 Testing of cubes

Specimens are usually sent to a laboratory for inspection and testing, but testing facilities are sometimes available on site. Reference should be made to the Standards for full details of requirements for both the testing machine and the testing procedure, but some of the important points relating to testing procedure for moulded cubes may be summarized as follows:

(1) The cube should be stored in water and tested immediately on removal from the water. Surface water, grit and projecting fins should be removed and the dimensions and weight noted.
(2) The bearing surface of the testing machine should be wiped clean and the cube should be placed in the machine in such a way that the load is applied to faces other than the top and bottom of the cube as cast. The axis of the cube must be carefully aligned with the centre of thrust of the machine.

(3) The load must be applied without shock and increased continuously at a rate of approximately 15 N/mm^2 (2200 lbf/in^2) per minute until no greater load can be sustained. The maximum load applied to the cube is recorded. The appearance of the concrete and any unusual features in the type of failure should be noted.

(4) The compressive strength is recorded to the nearest 0·5 N/mm^2 (50 lbf/in^2).

(5) It is important to maintain testing machines in good working condition and to see, for example, that the spherical seating can move correctly. The seating must move freely as the slack in the machine is taken up but must then lock and remain rigid until the cube fails; otherwise low failure loads will be recorded, and the shape of the failure will be one-sided. The form of failure should always be noted because an unusual shape of failure surface may indicate a defective machine.

11 INSPECTION AFTER CONSTRUCTION

The strength and durability of concrete structures is very much dependent upon control and inspection during construction as so many of the structural components are covered on completion. It is necessary, however, to inspect and assess concrete structures after completion. Methods of testing concrete after casting can be summarised as: —

visual inspection
extraction of cores
rebound hammer test
ultrasonic test
gamma-ray test
electromagnetic covermeter test
load tests.

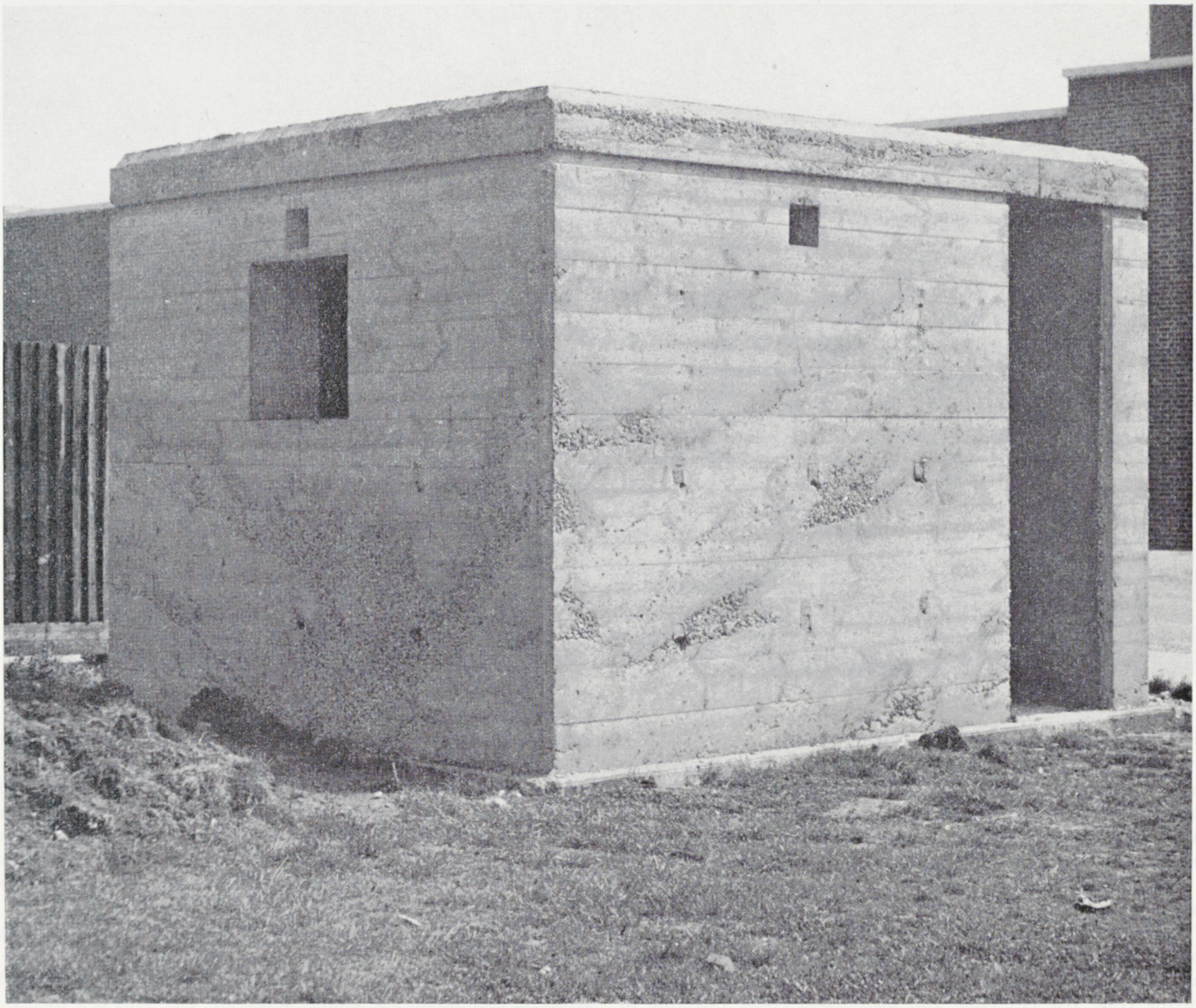

FIG. 7

Honeycombing, a result of poor compaction.

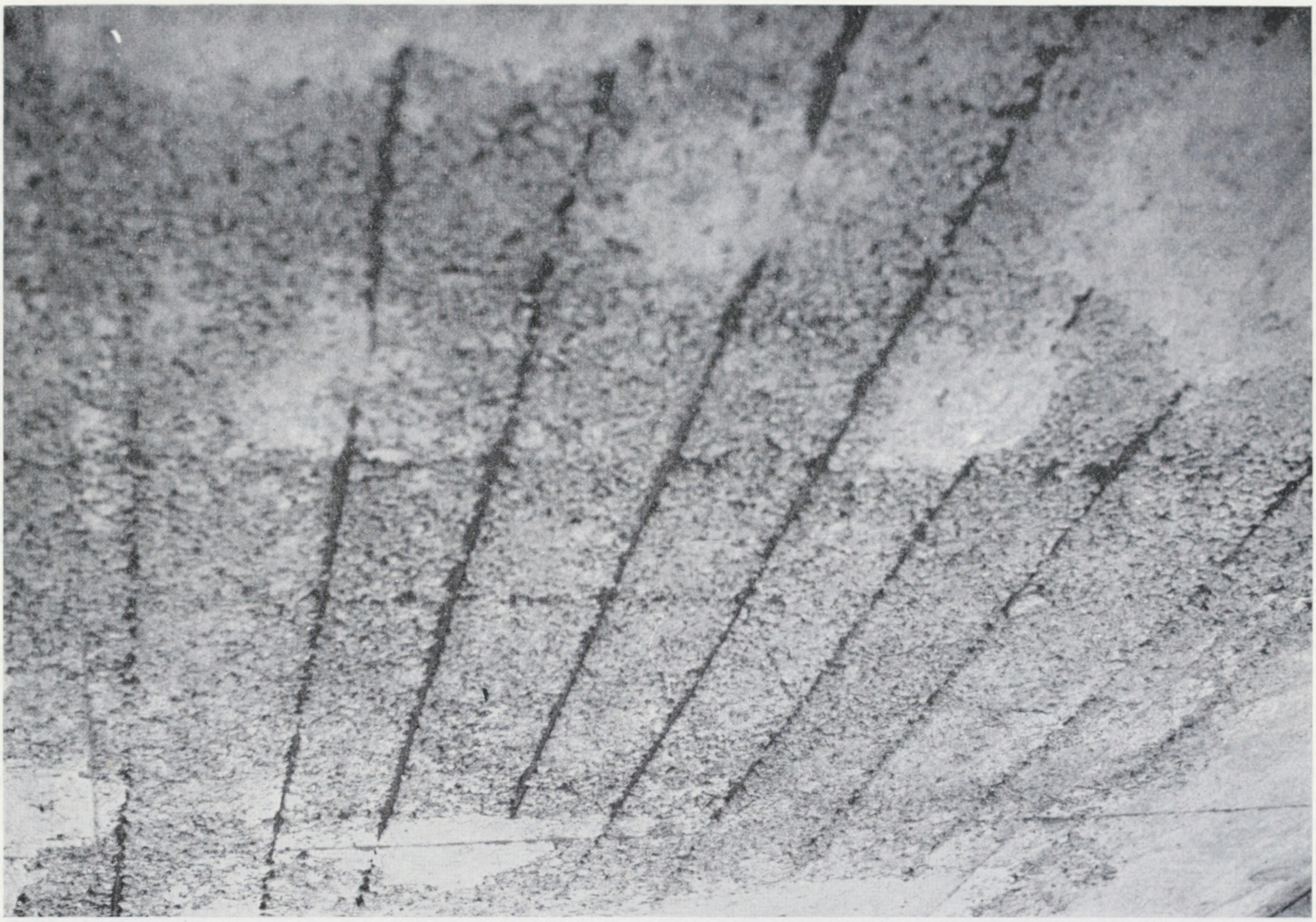

FIG. 8

Honeycombing, a result of poor compaction.

11.1 **Visual inspection of concrete structures**

If well-made formwork has been properly erected, and concrete of proper workability has been well compacted during placing, concrete structures require no further treatment after forms have been removed except for the possible filling of form tie holes.

Defects may occur at some time during construction which require repair or in more serious cases complete replacement of the defective member. The common defects in concrete construction are honeycombing (Figs. 7 and 8), grout loss at formwork joints (Fig. 9), poorly made construction joints, crazing (Fig. 10), cracking, and insufficient cover resulting in exposed reinforcement (Fig. 11).

Local honeycombing will not seriously affect the short term strength of the structure and the main concern with this defect is the corrosion of the embedded reinforcement and prestressing steel. The defective concrete should be cut out to a depth of at least 25 mm or until solid concrete is reached, the edges being cut perpendicular to the surface or, if possible, with a small undercut. The patching concrete should be a similar composition to the original concrete and well compacted into the void after previously saturating the area with water.

Another method of replacing defective concrete or building up the thickness of concrete where there is insufficient concrete cover to the reinforcing or prestressing steel, is a sprayed concrete technique, e.g. gunite, shotcrete, etc. This is a specialised process and should only be carried out by competent operatives experienced in the technique.

FIG. 9

Grout loss at shutter joints.

FIG. 10

Crazing on concrete cast against a smooth polished form face.

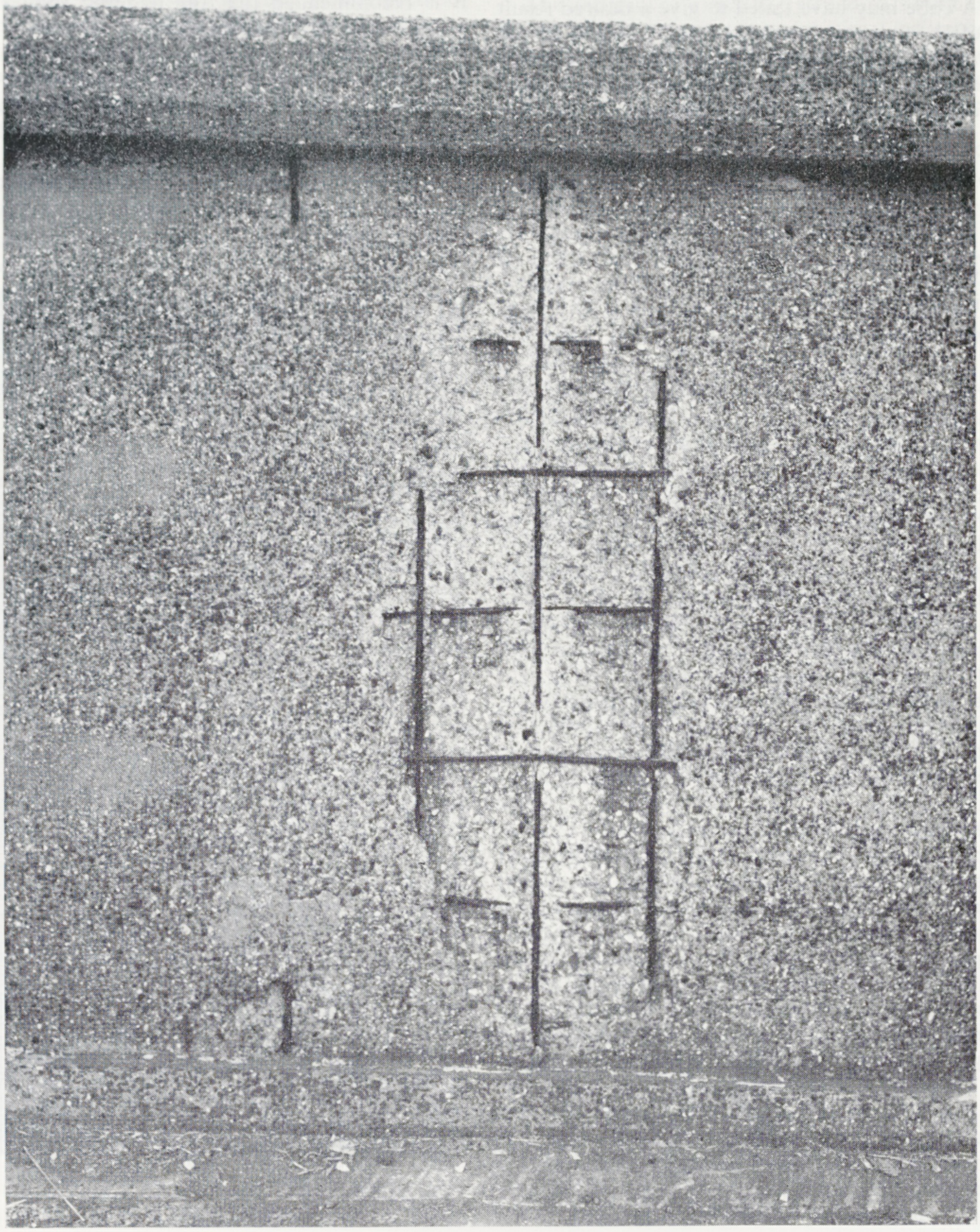

FIG. 11

Insufficient cover resulting in exposed reinforcement.

11.2 Test cores

Core tests are extracted for assessing the quality of hardened concrete in situ. Not only can the compressive strength of the concrete be assessed, but a description of the aggregate, including maximum size, shape, surface texture and type, can be obtained and the degree of compaction noted. The size, position, spacing and cover to reinforcement can often be checked though it is, of course, inadvisable to cut through main steel in structural members.

The usual diameter of a core is 150 mm (6 in) or 100 mm (4 in), and it is desirable to obtain a specimen that is roughly twice as long as its diameter. In thin members, or where reinforcement is congested, smaller cores may be necessary but these give less reliable strength test results. Core cutting is a skilled operation, usually performed by specialist sub-contractors.

11.3 Testing of cores

Core tests are frequently used when a cube test has proved unsatisfactory. A cube may have failed to give a desired result because of a defect in the testing procedure, in which case it is usual to examine the concrete in the structure in an attempt to assess its properties. Core tests need careful interpretation because the strength of a core is dependent on:

- quality of concrete;
- degree of compaction;
- location in the structure;
- curing;
- method of cutting;
- preparation of specimen;
- testing procedure.

In contrast, the cube test, if properly performed, uses a standardized preparation and testing procedure in an attempt to eliminate all variables except that of concrete quality. Concrete in a structure cannot be expected to have had the same treatment as a laboratory specimen. Accordingly, cores give very variable results and the equivalent cube strengths are usually lower than the measured cube strength of the concrete.

Because of the variability of the core test, more cores than cubes are needed to form a reasonable opinion as to the acceptability of concrete. It is also unwise to condemn the concrete in a pour on the basis of core tests unless cores from a part of the structure that has been found acceptable have also been tested to serve as a comparison.

Cores must be capped to give end surfaces that are plane, parallel and at right angles to the axis. The length and diameter of the core are measured because they are necessary for the calculation of strength. Core tests need the facilities of a laboratory.

The compressive strength of each core is calculated by dividing the maximum load by the cross-sectional area calculated from the average diameter. A correction factor, dependent upon the length/diameter ratio of the specimen after capping, is then applied. The product of this correction factor and the measured compressive strength is known as the corrected cylinder strength, this being the equivalent strength of a cylinder having a length/diameter ratio of 2. The presumed equivalent cube strength of the concrete is estimated by multiplying the corrected cylinder strength by 1·25, to allow for the difference in shape. This will give the equivalent cube strength of the concrete in the structure; because of differences in compaction, curing, age and other factors it cannot be taken as the strength which would have been attained by the concrete in a standard cube test.

11.4 Rebound hammer test

There are two types of rebound or Schmidt hammer, the P type and the N type. The P type is based on a pendulum and it is therefore unsuitable for use on soffits. The N type (Fig. 12) may be used on any inclination of surface because the reading depends on the action of a spring.

Rebound hammers give an approximate indication of concrete strength provided that users have prepared their own calibration charts by recording, as part of their quality control routine, the results of regular tests with the hammer on cubes and units made from the same concrete. To standardize readings as far as possible it is recommended that they should be taken on cubes held in the testing machine under a stress of at least 7 N/mm^2 (1000 lbf/in^2). Since readings taken on a trowelled face are often higher than those on a moulded face, it is recommended that the hammer should be used on the vertical face of the cube as cast.

When making the tests on units, special care must be taken to bed them firmly against the impact of the hammer, especially if they are relatively light. Readings should not be taken within 25 mm (1 in) of an edge. Results should be the average of at least nine individual readings.

A rebound hammer may also be useful for indicating the variability of concrete in a structure as a guide to the significance of a core test.

FIG. 12
The Schmidt hammer in use.

11.5 Ultrasonic test

Ultrasonic equipment, such as the 'Pundit', is now portable and consistent in its behaviour. Like the rebound hammer, it may be used for assessing the variability of concrete in a structure even though no absolute measure of strength may be obtained. If a calibration chart is constructed to show the relation between the readings on concrete components and the readings on cubes made from corresponding batches of concrete, such charts may be used to obtain approximate indications of the strength of concrete in the components.

11.6 Gamma-ray test

Gamma radiography may be used to test for the presence of local voids in concrete, the location and size of reinforcement and the efficiency of the grouting of cable ducts in post-tensioned prestressed concrete. Special precautions are necessary to avoid contamination from the radioactive source.

11.7 Electromagnetic covermeter

The covermeter is a non-destructive method of locating the depth and direction of reinforcement in hardened concrete. It is a portable electromagnetic instrument. Recent models have a work range of up to 100 mm (4 in) cover and operate from dry cells. The instrument consists of a searcher unit connected to the main unit containing the cells, circuits and indicator scale. When the instrument is switched on, or switched from one scale to another, it is necessary to place the searcher unit well away from any steel and to set the needle to the upper end of the scale by adjusting the 'zero set'. It is best to aim the search unit at the sky when doing this, but not if this means pointing it at scaffolding or other steelwork. The searcher unit is then placed on the surface of the concrete and, if any steel exists below the surface within a depth of 100 mm (4 in), the needle will move. The searcher unit is rotated and moved across the surface until the minimum reading is obtained; this reading indicates the cover to the steel, and the direction of the searcher unit indicates the direction of the steel.

The accuracy likely to be obtained on the average site is about $\pm$ 15 per cent or $\pm$ 5 mm ($\frac{3}{16}$ in), whichever is the greater, when used to detect mild steel bars of not less than 10 mm ($\frac{3}{8}$ in) diameter. Where bars of smaller diameter or of high-tensile steel are being detected, there is a tendency for the covermeter to indicate covers slightly greater than is actually the case.

If there is any doubt about the accuracy of the instrument, it can be calibrated easily by placing the searcher unit in line with a steel bar at a known distance away, say 25 mm (1 in), and checking the reading on the scale.

11.8 Load testing of structure

Loading tests on a completed structure should be made if required by the specification or if there is reasonable doubt as to the adequacy of the strength of the structure. Such tests need not be made until the expiry of 56 days of effective hardening of the concrete.

In such tests, the structure should be subjected to a superimposed load equal to one and a quarter times the specified superimposed load used for design, and this load should be maintained for a period of 24 hours before removal. During the test, struts strong enough to take the whole load should be placed in position leaving a gap under the members.

If within 24 hours of the removal of the load, the structure does not show a recovery of at least 75 per cent of the maximum deflection shown during the 24 hours under load, the test loading should be repeated. The structure should be considered to have failed to pass the test if the recovery after the second test is not at least 75 per cent of the maximum deflection shown during the second test.

If during the test, or upon removal of the load, the structure shows signs of weakness, undue deflection or faulty construction, it should be reconstructed or strengthened as necessary.

12 ACCURACY OF CONSTRUCTION

The accuracy with which a structure is built can be influential in its safety, but at the same time insistence upon an unwarranted degree of accuracy can be expensive. No measurement is absolutely accurate. All measuring equipment is only accurate to a particular tolerance, so accuracy should be related to the means of measurement.

If accuracy is to be achieved it must be demanded. Every operative and supervisor should know what tolerances are specified and how they are to be satisfied. If they are given in the specification but are not printed on the drawings, they should be written onto the drawings so that everyone on site is aware of them. If tolerances are nowhere specified they should be requested from the designer, and the means for achieving and checking them should be discussed with him.

12.1 Sources of inaccuracy

Inaccuracy can occur at a number of stages during the building process. The situation is most troublesome with precast concrete where construction is in three stages: manufacture, setting out and erection. (In in-situ work, manufacture and erection occur simultaneously.) If the setting out is inaccurate, progressive inaccuracy follows. If erection is dimensioned from previous work, progressive inaccuracies will also occur. Most care is needed when transferring dimensions up a building as it is built, because it is difficult to verify visually that walls and columns are accurately aligned when they are separated by a floor slab.

Problems with fit occur when components are manufactured off the site. Precast concrete cladding, windows, door sets, etc., all have to fit prepared openings in the structure. Tolerances for these situations must be carefully considered at the design stage.

12.2 Setting out

In normal construction work, setting out is the most accurate measuring operation undertaken. Centre lines or base lines should be established on site, from which intermediate measurements can be taken. The pegs or other markers which define these lines must be protected from damage during construction and must not, of course, be subject to ground movement during construction operations. It may be necessary to relate the base line to points outside the site boundary.

12.3 Precast components

Designers should provide means for the accurate adjustment of the position, level and plumbness of every component on site, and it should be made as easy as possible for operatives to work accurately. Positions for precast components should be clearly marked out, on centre lines or face positions, in such a way that some part of the line can still be seen after the component is fixed. If centre lines are being used, they should also be marked clearly on the relevant faces and edges of every component. Simple metal 'go/no-go' gauges should be made for the quick checking of joint widths, etc. Precasting has the advantage that the manufacturing accuracy of components may be checked off site and any necessary adjustments made more easily than if they had to be made in situ.

12.4 In-situ construction

Accuracy of in-situ construction is completely dependent on the formwork. Allowance must be made for the deflection of the forms and falsework under the loading of the wet con-

crete. With conventional timber formwork, adjustment of dimensions is easily made, but very fine tolerances are not possible because of the moisture movement to which timber is subject. A particular problem with in-situ concrete is to ensure that suspended floor slabs are of the correct thickness.

13 BSI PUBLICATIONS

13.1 British Standards for cements

(1) BS 12 *Portland cement (ordinary and rapid-hardening)*
- Part 1: 1958 Imperial units
- Part 2: 1971 Metric units

(2) BS 146 *Portland-blastfurnace cement*
- Part 1: 1958 Imperial units
- Part 2: 1973 Metric units

(3) BS 1370 *Low heat Portland cement*
- Part 1: 1958 Imperial units
- Part 2: 1974 Metric units

(4) BS 4027 *Sulphate-resisting Portland cement*
- Part 1: 1966 Imperial units
- Part 2: 1972 Metric units

(5) BS 4246 *Low heat Portland-blastfurnace cement*
- Part 1: 1968 Imperial units
- Part 2: 1974 Metric units

(6) BS— *Masonry cement* (in course of preparation)

13.2 British Standards for aggregates

(7) BS 877 *Foamed or expanded blastfurnace slag lightweight aggregate for concrete*
- Part 1: 1967 Imperial units
- Part 2: 1973 Metric units

(8) BS 882, 1201 *Aggregates from natural sources for concrete (including granolithic)*
- Part 1: 1965 Imperial units
- Part 2: 1973 Metric units

comprising:
- BS 882 *Coarse and fine aggregates from natural sources*
- BS 1201 *Aggregates for granolithic concrete floor finishes*

(9) BS 1047 *Air-cooled blastfurnace slag coarse aggregate for concrete*
- Part 1: 1952 Imperial units
- Part 2: 1974 Metric units

(10) BS 1165: 1966 *Clinker aggregate for concrete*

(11) BS 1198–1200: 1955 *Building sands from natural sources*

(12) BS 3797: 1964 *Lightweight aggregates for concrete*

(13) BS 4619: 1970 *Heavy aggregates for concrete and gypsum plaster*

13.3 British Standards for admixtures

(14) BS 1014: 1961 *Pigments for cement, magnesium oxychloride and concrete*

(15) BS 3892: 1965 *Pulverized-fuel ash for use in concrete*

(16) BS 5075 *Concrete admixtures*
- Part 1: 1974 Accelerating admixtures, retarding admixtures and water-reducing admixtures

13.4 British Standards for reinforcing steel

(17) BS 4449: 1969 *Hot rolled steel bars for the reinforcement of concrete*

(18) BS 4461: 1969 *Cold-worked steel bars for the reinforcement of concrete*

(19) BS 4466: 1969 *Bending dimensions and scheduling of bars for the reinforcement of concrete*

(20) BS 4482: 1969 *Hard-drawn mild steel wire for the reinforcement of concrete*

(21) BS 4483: 1969 *Steel fabric for the reinforcement of concrete*

13.5 British Standards for prestressing steel

(22) BS 2691: 1969 *Steel wire for prestressed concrete*

(23) BS 3617: 1971 *Seven-wire steel strand for prestressed concrete*

(24) BS 4757: 1971 *Nineteen-wire steel strand for prestressed concrete*

(25) BS 4486: 1969 *Cold-worked high-tensile alloy steel bars for prestressed concrete*

13.6 British Standards for testing

(26) BS 410: 1969 *Test sieves*

(27) BS 812: 1967 *Methods for sampling and testing of mineral aggregates, sands and fillers*

(28) BS 1881 *Methods of testing concrete*
- Part 1: 1970 Methods of sampling fresh concrete
- Part 2: 1970 Methods of testing fresh concrete
- Part 3: 1970 Methods of making and curing test specimens
- Part 4: 1970 Methods of testing concrete for strength
- Part 5: 1970 Methods of testing hardened concrete for other than strength
- Part 6: 1971 Analysis of hardened concrete

(29) BS 3148: 1959 *Tests for water for making concrete*

(30) BS 3681 *Methods for the sampling and testing of lightweight aggregates for concrete*
- Part 1: 1963 Imperial units
- Part 2: 1973 Metric units

(31) BS 4408 *Recommendations for non-destructive methods of test for concrete*

Part 1:1969 Electromagnetic cover measuring devices

Part 2:1969 Strain gauges for concrete investigations

Part 3:1970 Gamma radiography of concrete

Part 4:1971 Surface hardness methods

Part 5:1974 Measurement of the velocity of ultrasonic pulses in concrete

(32) BS 4550 *Methods of testing cement*

Part 2:1970 Chemical tests

13.7 **Other British Standards**

(33) BS 1305:1974 *Batch type concrete mixers*

(34) BS 1926:1962 *Ready-mixed concrete*

(34a) BS 4251:1974 *Truck type concrete mixers*

13.8 **Codes of Practice**

(35) CP 110 *The structural use of concrete*

Part 1:1972 Design, materials and workmanship

(36) CP 114 *Structural use of reinforced concrete in buldings*

Part 2:1969 Metric units

(37) CP 115 *The structural use of prestressed concrete in buildings*

Part 2:1969 Metric units

(38) CP 116 *The structural use of precast concrete*

Part 2:1969 Metric units

(39) CP 221:1960 *External rendered finishes*

(40) CP 2007 *Design and construction of reinforced and prestressed concrete structures for in storage of water and other aqueous liquids*

Part 1:1960 Imperial units

Part 2:1970 Metric units

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Discussion

on

Mr. D. L. Schonhut's Paper

CONCRETE CONSTRUCTION

Paper No. 5. Session 1975-76

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Hon. Sec. A. Wardle
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Discussion on Mr. D. L. Schonhut's Paper

CONCRETE CONSTRUCTION

Mr. G. P. SMEDLEY

Mr. Schonhut is to be congratulated on an excellent paper and lecture which were prepared at short notice to fill a gap in the programme of the Technical Association. The paper contains a wealth of information on concrete construction which is of value to colleagues in Head Office and at Outports.

In the past Surveyors have had occasionally to deal with certain types of concrete constructions. It was only with the advent of large offshore gravity concrete structures that the Society engaged specialist civil engineers to deal with design appraisal, plan approval and surveys during construction and service.

With reference to design appraisal, these Surveyors developed techniques which were in advance of those of the industry, and are now widely recognized. Mr. Schonhut deals specifically with the quality control and assurance of reinforced and pre-tensioned concrete structures and is particularly competent in these areas.

Although the Surveyors who are specialists in concrete and soil mechanics are based in the Offshore Services Group, the Chief Surveyors insist rightly that their services are available to all departments in the Society having need of them. OSG is encouraged to run an 'open house' and colleagues in other departments are welcome to discuss problems of concrete constructions or foundations with those concerned with these disciplines.

Reinforced concrete is one of a number of composite materials. With changing economics of construction, these composites can be attractive to owners. In recent years the use of concrete has been extended to the construction of nuclear pressure vessels, ships, offshore structures, storage tanks, etc. For the benefit of colleagues, I should be pleased if Mr. Schonhut could give a little more information on practice for obtaining adequate reinforced concrete in relatively thin sections.

In dealing with the properties of concrete he gives limited data on permeability, fatigue strength, resistance to impact damage and spalling under repeated temperature transients. Each of these can be of major importance in many proposed applications. I should be pleased if he would provide further information for guidance in design.

Mr. J. G. BEAUMONT

I'm afraid I, like everyone else, have had very little time to study this paper in depth. Accordingly I would like at this stage to confine myself to main points. Speaking as a naval architect concerned basically with floating structures I must admit the use of concrete in this context has come up rather late in life and I'm not fully sure what's hit me yet. Since, no doubt, I am not alone in this situation, this paper gives an excellent information coverage of the elements of the subject and Mr. Schonhut is to be congratulated for his considerable efforts.

Since the revision of Chapter D of the Rules could well include rules for concrete ships in the near future, I will not touch upon problems of assessing the strength of concrete structures, such as characteristic strength, suitable minimum cover to reinforcement, etc.: rather I will concentrate on the practical aspects of construction under survey.

In his introduction Mr. Schonhut mentions quality assurance. In classification terms for steel ships this means that the Surveyor is totally satisfied that the constructor's quality control procedures are effective so that he, the Surveyor, can recommend to the Committee by virtue of his continuous monitoring of the system, that the finished structure is eligible for class. In the context of concrete construction a very fundamental point is raised here. Unlike steel construction, we are not dealing with the quality of a structure erected from material previously tested for strength and suitability. We are dealing with structures of which the very fabric is dictated by the control which is exercised during a few critical hours between preparation of components for mixing each batch and completion of pouring. Moreover, batch mixing is often virtually a continuous process on a large site. Construction usually proceeds quickly requiring full-time attendance of those responsible for quality. The decision whether to knock out a bad pour sometimes has to be made on the spot. Delay in decision is economically unacceptable especially in prestressed work.

The question raised by such a procedure is 'What is the Surveyor's proper role in all this'? Non-destructive testing is only possible to a limited extent so that if control of workmanship is poor during preparations for the pour, be it preparation of mixing, cleanliness and tightness of shuttering, accuracy of placement of shuttering, preparation of construction joints, or proper compaction, then the long-term adequacy of a marine structure could be forever in doubt. It is my own view that quality assurance is the correct approach but only after fully trained Surveyors have established beyond all doubt that the quality control of the sitework and recording techniques is beyond reproach. Until this has been established, full-time attendance of a trained Surveyor is considered vital. Having said this I must admit that it's not that simple. In the U.K. I understand that in civil engineering the client often contracts a consultant engineer to design the structure. The constructor is then contracted to build the structure under the supervision of a resident engineer appointed by the consultant engineer, the statutory inspection being a matter for the local Borough Surveyor or Clerk of Works. Where the constructor himself is the designer (and this I understand is frequently the case abroad) there is no resident engineer and full time inspection is arranged by the client. It will be interesting to see how the role of the Classification Surveyor develops in this new context. Meanwhile perhaps Mr. Schonhut could develop a little on quality assurance of concrete construction, as generally understood in the industry. However, before leaving this topic I would add that I've listened to 'old hands' in the U.K. construction business talking and have learned that one of the surest ways to find out how things are going on a site is to review the written dialogue between the resident engineer and the constructor. I doubt if outsiders are privileged to this information!

Turning now to a few details of the paper I wonder if Mr. Schonhut would agree that it is best to specify the number

of workability tests with respect to batch sizes? (Paragraph 5.3.6 refers.)

In marine work we are often dealing with thin heavily reinforced vertical wall sections having widely spaced horizontal construction joints. It is important to emphasize the length as well as the thickness of vibrators in these conditions. It is invariably at the base of such walls that good compaction is most difficult to achieve.

With reference to water bars (Section 5.6.3), in floating structures; would Mr. Schonhut agree that a very careful interface preparation at the construction joint together with tight shuttering and good compaction, is a better proposition than a water bar?

Cover is vitally important in marine work especially where minimum covers are adopted. Would Mr. Schonhut agree that a covermeter is an essential piece of equipment in this type of work, especially if the Surveyor's role is to be a quality assurance one rather than a direct inspection one?

Mr. B. Rapo

I would like to congratulate the Author on a most interesting and informative paper which was long overdue.

It is now almost two years since the Society completed an initial feasibility study on a large prestressed and reinforced concrete LNG carrier of 125 000 m^3 capacity. This study was restricted in scope to two fundamental aspects, namely, LOADING and STRUCTURAL RESPONSE within the cargo tank region. The third, equally important aspect, that of CONSTRUCTION embracing materials, testing, inspection and repair procedures was not at that time considered. With this paper we have an excellent opportunity to discuss in principle, some of the problems which are related to this subject. My initial reference will concern the mix design of the proposed LNG carrier.

In paragraph 2.2.2, describing the lightweight and manufactured aggregates, statement is made to the effect that 'all lightweight materials are relatively weak because of the porosity which gives them reduced weight'. While this statement on its own is perfectly correct, I would like to suggest that one should not interpret it in the sense that a 'weak' aggregate cannot produce a 'strong' end product, which is concrete. The water/cement ratio is relatively the most important item in determining concrete strength. It is only the ceiling strength which may be affected by properties of the aggregate.

If one should have to define the desirable features of an ideal lightweight aggregate to be used for construction of ships of a commercially acceptable deadweight/displacement ratio, the following features are considered to be of interest:

Low specific gravity (preferably below 1·5). The expanded shale aggregate marketed under name BERWILIT is stated to have a specific gravity of 1·64 leading to a gross density of hardened concrete of the order of 1·95. It may be of interest to note that reducing the density of concrete from, say, 2 to 1·9 is equivalent to an increase in cargo deadweight/displacement of about 2300 tonnes for the LNG carrier mentioned above.

Water absorption rate not exceeding 12 per cent for any size of aggregate. This is possible if all the particles, during manufacturing, are given a semi-impervious glazed surface.

Good abrasion resistance. This is, of course, difficult to elaborate on in terms of a simple numerical index. Abrasion test requirements prescribed by ASTM should be basically interpreted on a comparative basis.

Moderate shrinkage.

Good durability.

I would like to refer now to Table 7, which specifies the thickness of, presumably, impermeable and uncracked concrete cover recommended for prestressed and reinforced concrete constructions. It can be observed that, for grade 30 concrete, the required cover is of the order of 50 mm if the construction is exposed to sea water. If this is intended to serve as a guide in marine applications to ensure adequate protection against corrosion of the reinforcing steel, then those of us who were intimately involved in the formulation of the Society's policy in the ship field, should be very concerned. Our attitude was to accept 25 mm as the thickness of protective layer over the reinforcements. The greater the thickness of this layer, the greater will be probability of having local cracking under secondary flexural deformation. There is evidence that concrete hulls built during World War II were designed with a coverage of about 25 mm and they have shown good long-term endurance.

Fig. 11 is in a way terrifying. Is the exposure of wire reinforcement due to 'insufficient cover' or due to possible rubbing against an object not visible in the photograph?

Paragraph 4.2.1 deals with the problem of permeability of concrete structures. This aspect is of particular interest in the design of either water-retaining structures or structures whose function is to keep water out, for example ships. Permeability in the latter case is even more complex since it is not related only to possible movements of water between gel and capillary pores but to water vapour penetration. Apart from keeping the water/cement ratio down to a level which would still ensure satisfactory workability, proper grading of aggregates and making provision for moist-curing which will enable a maximum rate of hydration, it would be of interest to know if there is anything else that can be done at the design stage to ensure effective water- and air-tightness. Are there any corrosion inhibiting admixtures worth considering which could be recommended by the Author?

Mr. T. Lindsay

I would like to begin with a tribute to the value of this paper. The Author has covered the subject from A to Z and has left very few stones unturned, this being particularly true in the sections of most interest to the Surveyor engaged at the building site. Mr. Schonhut's paper could serve as a blueprint for the formulation of guidance notes, although certain areas no doubt would change depending on the type of structure.

Dispensing of the admixtures, as the paper rightly states, necessitates a greater degree of control than normally applies to the other constituents—the dosage can be of the order of 300 parts per 10^6. To obtain optimum performance it is essential that the admixture be uniformly distributed throughout each batch of concrete, and therefore it is usual for them to be supplied as water solutions or as water-soluble powders for addition to the mixing water.

As a point of interest I understand that in the U.S.A. it is standard practice to deliver ready mix concrete air-entrained.

Referring to Table 3, may I suggest that graphs best illustrate the growth in strength of the concrete with age as hydration of the cement continues. Fig. D 1 shows a set of graphs and is a guide to strength development at 20°C for water-cured natural aggregate concrete, in addition it shows typical values for the water/cement ratio.

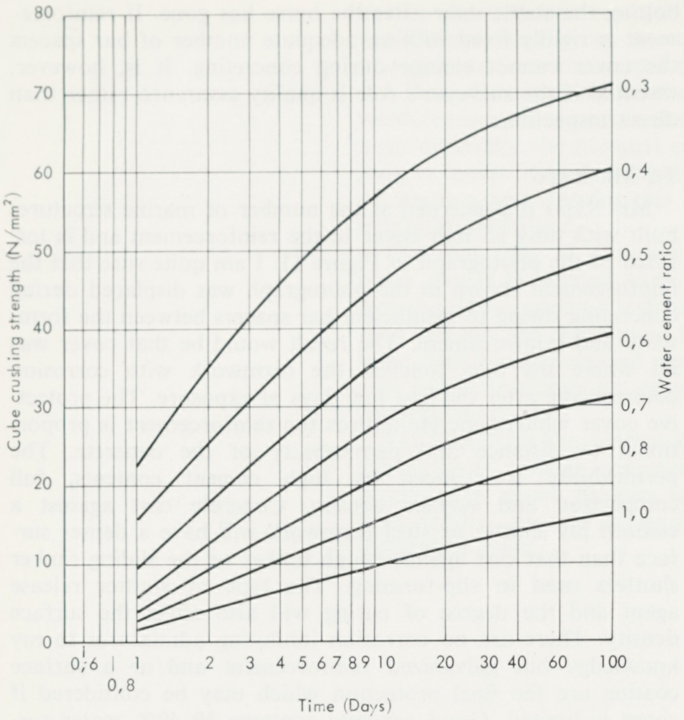

FIG. D 1

Guide to strength development of 20°C water-cured natural aggregate concrete.

I would like to raise one question which is not relevant strictly to the title of the paper but is, I believe relevant to the way the Author has approached his subject. The repair of damaged tendons in prestressed concrete has, from time to time over the past two years, been the subject of much discussion in the Hull Structures Department, and perhaps Mr. Schonhut would consider letting us have a sentence or two in his written reply, on structures known to him that have been successfully repaired with the prestress fully restored, also his own view on the subject.

MR. J. J. GOODWIN

I should like to compliment the Author on a fine paper and an interesting lecture.

Paragraph 2.2.1.5 Marine aggregates

The Author states that shell and salt, which may be present in marine aggregates, are not harmful in reinforced concrete, but marine aggregates should not be used for prestressed concrete. Would the Author explain why marine aggregates should not be used for prestressed concrete.

Table 2 Characteristic strengths

The Author gives recommended grades of concrete with appropriate uses. I would suggest that concrete with a characteristic strength of at least 40 N/mm^2 is desirable to provide adequate durability in marine applications. In the case of concrete vessels constructed with natural aggregates, where weight is a premium, the use of concrete of a lesser grade than 40 may be expected to be uneconomical. I feel therefore that this table could be misleading to the Surveyor and would appreciate the Author's comments.

Table 7 Exposure and cover

Conditions of exposure are categorized as mild, moderate, severe and very severe, however the descriptions accompanying these conditions of exposure do not appear to be appropriate to the marine environment. Would the Author express his opinion as to the nominal cover required in Grade 40 concrete for those parts of a reinforced concrete vessel which are: —

(a) Totally submerged in sea water, e.g. the vessel's keel.
(b) Partially submerged in sea water, e.g. sides of the vessel.
(c) Not submerged but subject to sea water spray, e.g. deck, etc.

Paragraph 10.1.3 Sampling of concrete

Would the Author give a guide to the number of samples required in relation to the volume of concrete placed and the number of batches delivered?

Would the Author recommend the taking of samples for cube testing at both 7 days and 28 days or does he feel that it is sufficient to test at 28 days only?

Finally, although perhaps outside the scope of the Author's paper, I would value his opinion on the following matter. There appears to be some dissent in the construction industry that CP 110 Section 6 imposes a too severe criterion on cube tests in that the average strength of any four consecutive test cubes should exceed the characteristic strength by 0·5×the current margin. Would the Author comment on this requirement?

AUTHOR'S REPLY

TO MR. SMEDLEY

In reply to Mr. Smedley's request for more information on relatively thin reinforced concrete sections I would ask all designers, particularly in water retaining structures, to think seriously of the practicalities of construction. It is not difficult to design thin concrete shells where the concrete is always in compression, it is, however, difficult to place and compact concrete in a confined space. In walls the concrete is gravity fed into the top of the formwork, it partially segregates as it falls to the bottom of the shutters where it remixes and flows around the reinforcement and numerous other obstructions. Compacting the concrete to remove all entrapped air by use of poker vibrators is difficult in a confined space and my advice therefore is to avoid thin sections, i.e. less than 300 mm, to use high workability concrete by incorporating plasticizers or super plasticizing admixtures, and to detail the reinforcement to minimize restriction to the flow of concrete.

The coefficient of permeability of concrete is dependent on the water/cement ratio, the cement content, full compaction and proper curing. The range of values is probably of the following orders: —

Good quality concrete	$K = 10^{-12}$ m/s
Normal quality concrete	$K = 10^{-10}$ m/s
Poor quality concrete	$K = 10^{-8}$ m/s

On the question of fatigue in composite, reinforced or prestressed concrete sections, the levels of stress in the steel are a greater percentage of the ultimate stress than for concrete. It is generally sufficient to consider the fatigue properties of the steel as controlling the fatigue performance of the structural element. Experiments on the fatigue performance of reinforced concrete saturated with sea water indicate that providing the normal design rules are applied, such as depth of cover and allowable crack widths, no corrosion fatigue should occur.

Resistance to impact damage is a difficult question to answer but in general the thickness of the section, the area of reinforcement and the amount of prestress are fundamental in the restriction of crack propagation in the damaged area.

Concrete will not spall under repeated temperature transients unless the temperature falls repeatedly below freezing point. Good quality concrete is resistant to sub-zero temperatures down to cryogenic temperatures but poor quality concrete will spall due to the disruptive forces of water freezing in the voids within the concrete mass. The resistance to sub-zero temperatures is improved by air entrainment but the concrete strength is reduced at the same time so careful control is required.

To Mr Beaumont

Mr. Beaumont asks if I can develop on the quality assurance of concrete construction. The quality assurance in concrete construction, as in many other fields, is a series of relatively minor but equally important items. The materials should be subject to routine quality control tests before delivery with adequate storage facilities after delivery. The inspection of site fabrication, i.e. formwork, fixing of reinforcement, prestressed cable ducts and embedded pipework, construction joints etc. is of paramount importance before concreting as it is extremely difficult to detect and expensive both in time and cost to correct any defect or inaccuracy when the concrete has been placed. The site production, transport, placing and compaction of concrete is monitored by visual inspection with workability tests on the concrete and sampling for cube tests taken at random. Finally, the protection of concrete during the hardening and curing process and the period which elapses before formwork is removed must be monitored.

On the question of workability tests, the water content or water/cement ratio of concrete is important and the workability test will monitor variations, but equally the experienced eye of a competent inspector or foreman is a great deal quicker and can often detect variations which may not be highlighted by the workability test, e.g. excessive or insufficient sand in a batch will increase the porosity of the concrete.

I do not consider the length of poker vibrators as important as ensuring that the concrete is distributed from the skip or pipeline evenly in layers 300–400 mm deep in thin, heavily reinforced vertical wall sections. Many problems are caused by depositing a whole batch or skip of concrete in one position and then trying to make the concrete flow horizontally by means of the poker vibrators.

I would agree that interface preparation is imperative at construction joints in floating structures but also think that water bars are necessary particulary in thin walls for, as Mr. Beaumont states, it is difficult to achieve good compaction at the base of walls.

A cover meter is a useful piece of equipment for checking the concrete cover to reinforcement, but this is rather like bolting the stable door after the horse has gone. If reinforcement is rigidly fixed with an adequate number of bar spacers the cover cannot change during concreting. It is, however, essential if the surveyor's role is quality assurance rather than direct inspection.

To Mr Rapo

Mr. Rapo is concerned at the number of marine structures built with only 25 mm cover to the reinforcement and is terrified of the photograph in Figure 11. I am quite sure that the reinforcement shown in the photograph was displaced during concreting owing to insufficient bar spacers between the formwork and reinforcement. The result would be that cover was nil where the bars touched the formwork with corrosion commencing after the first few days of exposure. The protective cover which concrete affords the reinforcement is proportional to distance and permeability of the concrete. The permeability is reduced by high cement contents, full compaction and surface density. Concrete cast against a smooth ply shutter or steel formwork will have a denser surface than that cast against rough timber or the sliding timber shutters used in slip-forming. The type of shutter release agent and the degree of curing will also affect the surface density. There are no corrosion inhibiting admixtures to my knowledge but galvanized reinforcement and/or a surface coating are the final protection which may be considered if cover is limited. Good concrete contains 30–40% water surplus to hydration requirements which, if allowed to dry out, leaves voids. Certain surface coatings, e.g. polyurethanes, sodium silicates, chlorinated rubber paints, bitumens, etc., are compatible with the high alkalinity of concrete, and will fill some of the voids and improve corrosion resistance, but the maintenance of surface coatings will be very expensive.

To Mr Lindsay

In reply to Mr. Lindsay's question, the repair of damaged tendons in prestressed concrete is only possible during the initial stressing stage. It is not unusual for some wires or even a strand to break during stressing and it is prudent to express the amount which will be acceptable before stressing commences. If the loss of area of prestressing steel is unacceptable the tendon should be de-stressed and replaced. The losses which can be accepted should be stipulated for a single tendon and for a group of tendons. The tendons are adequately protected after grouting and no damage should occur during the life of the structure. The problem of blocked ducts should be given thought in highly stressed areas and the provision of spare ducts or higher stressed cables used elsewhere if the loss of a duct is critical. I thank Mr. Lindsay for Fig. D.1 which I intended including in the paper but which I forgot to do in my haste. The graph clearly demonstrates the effect of water/cement ratio on strength.

To Mr Goodwin

Mr. Goodwin asks why marine aggregates should not be used for prestressed concrete. Maybe my statement is too strict as the draft B.S. Code for the fixed offshore platforms proposes to limit the chloride content of the aggregates to less than 0.1% by weight of the cement content of the concrete which in effect means that marine aggregates which are adequately washed with fresh water are suitable.

Table 2 is offered as a guide for general construction and I would agree that for a marine environment concrete should

have a characteristic strength of at least 40 N/mm^2. Specific applications require an assessment of the design, available materials and environmental data.

I have discussed earlier the question of concrete cover to reinforcement in a marine environment and feel that for long term durability the nominal cover should not be less than 40 mm. In areas of severe exposure to wetting and drying and/or freezing and thawing this cover should be increased by a minimum of 10 mm.

The frequency of sampling concrete for cube testing purposes should be related to the volume of pour but each project should be assessed individually. On small pours, say less than 100 m^3, the rate should be one set per 50 m^3 or less; on large continuous pours of several thousand cubic metres the rate could justifiably be increased to one set per 100 m^3 or possibly more.

The dissent in the construction industry to Section 6 of CP 110 is from smaller contractors who have minimal quality control. On major projects where continuous production of high quality concrete is required, the criterion should not be difficult to meet.

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